

**University of Southern Queensland**  
**Faculty of Engineering and Surveying**

**Design, Construction and Operation of the  
Floating Roof Tank**

A dissertation submitted by

Submitted by

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in fulfilment of the requirement of

**Course ENG 4111 and ENG 4112 Research Project**

towards the degree of

**Bachelor of Engineering (Mechanical Engineering)**

Submitted: 29<sup>th</sup> October 2009

# **ABSTRACT**

Storage tanks have been widely used in many industrial particularly in the oil refinery and petrochemical industry which are to store a multitude of different product with crude oil as one if it. There are different types of tank such as fixed roof tank, open roof tank, floating roof tank etc. Floating roof tank is which the roof floats directly on top of the product, with no vapour space and eliminating the possibility of flammable atmosphere.

There are various industrial code and standard available for the basic requirement for tank design and construction. Commercial software are also available in the market for the basic design, hence tank designer would rely wholly on the software without detail understanding. Despite of the various standard and code, there is limited procedure and rules in designing the floating roof which result lots of floating roof failure and caused injuries and fatalities accident. Design and safety concern has been a great concern for the increasing case of fire and explosion due the tank failure.

The main objective of this project is “HOW TO DESIGN A NEW FLOATING ROOF TANK”. The aim of this project is to develop basic rules and procedures, highlighting the concerns in designing, construction and operation of a floating roof by taking an existing Oil Development Project with it’s readily available information as a base, to design the tank, and identify the problematic and lesson learnt throughout the project.

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I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

KUAN SIEW YENG

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# Acknowledgment

This research was carried out under the principal supervision of Dr. Harry Ku and the co-supervisor is Dr. Talal. I would like express my great appreciation toward them for their kind valuable assistance and advice through out the project.

Beside that, I would like to thanks the library of Technip Malaysia which had provided me a lot of handfull information and reference book as this project requires lot of reference and international code.

# **TABLE OF CONTENT**

<b><u>CONTENTS</u></b>	<b><u>PAGE</u></b>
ABSTRACT	i
ACKNOWLEDGMENT	iv
LIST OF FIGURES	xi
LIST OF TABLES	xvi

## **CHAPTER 1:INTRODUCTION**

1.1 Rationale	1
1.2 Research Goal	2
1.2.1 Project Aims	2
1.2.2 Project Objective	2
1.3 Research Methodology	
1.3.1 Literature Review	3
1.3.2 Case Study	3
1.3.3 Product Enquiries	3
1.3.4 Design Approach	3
1.3.5 Consequential Effect of the Design Failure	4
1.3.6 Special Design and Construction	4

## **CHAPTER 2:LITERATURE REVIEW**

2.1 Introduction	5
2.2 Type of Storage Tank	7
2.2.1 Open Top Tank	7
2.2.2 Fixed Roof Tanks	8
2.2.3 Floating roof Tanks	9

2.3	Design Code and Standard	10
2.4	Floating Roof Tank	11
2.4.1	History and Introduction	11
2.4.2	Principles of the Floating roof	11
2.4.3	Advantages of the Floating Roof Storage Tank	13
2.5	Design Data Overview	13
2.6	Process Description and Requirements	15
2.7	Process Description and Design Consideration	16
2.8	Material Selection and Corrosion Assessment	19
2.8.1	CO <sub>2</sub> Corrosion	19
2.8.2	Carbon Dioxide Corrosion Modeling	21
2.9	Mechanical Selection of Carbon Steel Grade	22
2.10	Mechanical Design	25
2.11	Tank Shell Design Method as Per API 650	26
2.11.1	Calculation of thickness by 1-Foot Method	26
2.11.2	Calculation of thickness by Variable-Design-Point Method	27
2.11.3	Calculation of thickness by Elastic Analysis	28
2.12	Mechanical Design consideration	28
2.13	Bottom Plate Design	30
2.13.1	Vertical Bending of Shell	30
2.14	Floating Roof design	31
2.15	Special Consideration	32
2.15.1	Soil Settlement	32
2.15.2	Seismic Design for Floating roof	33
2.16	Failure Mode Due to Seismic Effects on Floating Roof Tank	34
2.17	Fitting Design and Requirement	36
2.18	Typical Fitting and Accessories for Floating Roof	37

2.18.1	Roof Seal System	37
2.18.2	Support Leg	38
2.18.3	Roof Drain System	39
2.18.4	Vent – Bleeder vents	43
2.18.5	Centering and Anti-Rotation Device	44
2.18.6	Rolling Ladder and Gauger Platform	44
2.19	Fire Fighting System and Foam Dam	44

### **CHAPTER 3:TANK DESIGN**

3.1	Introduction	46
3.2	Shell Design	46
3.2.1	Longitudinal Stress	47
3.2.2	Circumferential Stress	48
3.2.3	Longitudinal Stress versus Circumferential Stress	49
3.2.4	Circumferential Stress Thickness Equation and 1-Foot Method	49
3.2.5	Shell Design Thickness calculation	50
3.2.6	Top Stiffener and Intermediate Wind Girder Design	
3.2.6.1	Top Stiffener/ Top Wind Girder	51
3.2.6.2	Intermediate Wind Girder	54
3.2.7	Overturning Stability against Wind Load	57
3.2.8	Seismic Design	60
3.2.8.1	Site Geometry Design Data for Seismic Design	62
3.2.8.2	Overturning Stability	62
3.2.8.3	Design Spectral Accelerations	64

3.2.8.4	Parameter required for Seismic Design	65
3.2.8.5	Effective Weight of Product	69
3.2.8.6	Center of Action for effective Lateral Force	71
3.2.8.7	Ring Wall Moment	72
3.2.8.8	Base Shear Force	72
3.2.8.9	Resistance to Overturning	74
3.2.8.10	Anchorage Design	77
3.2.8.11	Freeboard	78
3.2.8.12	Seismic design Summary	79
3.3	Roof Design	80
3.3.1	Roof type Selection	80
3.3.2	Pontoon and Center deck Design	81
3.3.2.1	Roof Stress Design	82
3.3.2.2	Effect of large Deflection on Center Deck	83
3.3.2.3	Pontoon Stability – Pontoon Ring Design	86
3.3.3	Fitting and Accessories Design	89
3.3.3.1	Roof Seal System	90
3.3.3.2	Roof Seal Material	95
3.3.3.3	Roof Support Leg	96
3.3.3.4	Venting System	98
3.3.3.4.1	Operation of Bleeder Vent	98
3.3.3.4.2	Bleeder Vent Design	101
3.3.3.5	Roof Drain System	104
3.3.3.5.1	Articulated Piping System	105
3.3.3.5.2	Flexible Drain Pipe System	107

3.3.3.5.3	Drain System Selection	109
3.3.3.5.4	Drain Pipe Design	110
3.3.3.6	Rolling Ladder & Gauger Platform	112
3.3.3.7	Fire Fighting System and Foam Dam	113

## **CHAPTER 4:TANK CONSTRUCTION**

4.1	Introduction	116
4.2	Foundation	117
4.3	Bottom Plate Placement	118
4.4	Shell Erection	121
4.5	Tank Testing	
4.5.1	Tank Bottom Testing	123
4.5.2	Tank Shell Testing	123
4.5.3	Floating Roof Testing	125

## **CHAPTER 5:SPECIAL CONSTRUCTION**

5.1	Design consideration	
5.1.1	Design Consideration of Foundation	127
5.1.2	Design consideration on Tank Shell	129
5.2	Construction Consideration	
5.2.1	Nominal Diameter Versus Inside Diameter	130.
5.2.2	Plate Square-ness	130
5.2.3	Wind Damage	131
5.3	Testing Consideration	

5.3.1	Hydrotest/ Water Test	131
<b>CHAPTER 6: CONCLUSION</b>		132
<b>REFERENCE</b>		134
<b>APPENDIX A:</b>	Project Specification	A1
<b>APPENDIX B</b>	Design Calculation	B1

## LIST OF FIGURE

## PAGE

Figure 1.1: Fire and explosion incidents in the tanks	6
Figure 1.2: Types of storage tank	7
Figure 1.3: Types of Fixed Roof Tanks	8
Figure 1.4: Single Deck Pontoon Type Floating Roof	9
Figure 1.5: Double Deck Type Floating Roof	10
Figure 1.6: Single Deck Floating Roof Tank	12
Figure 1.7: Double Deck Floating Roof Tank	13
Figure 1.8: Storage Tank Capacities and Levels	15
Figure 1.9: Schematic Sketch of the Stabilised Condensate Tank	17
Figure 1.10: Impact Test Exemption Curve	23
Figure 1.11: Tank Exploding	26
Figure 1.12: Loading Diagram on a Tank Shell	29
Figure 1.13: Rotation of the shell-to-bottom connection	30
Figure 1.14: Single Deck Roof Sagged with Flooding Rain Water	31
Figure 1.15: Floating roof overtopped	34
Figure 1.16: Pontoon buckling	34
Figure 1.17: Diamond buckling (slender tanks)	35
Figure 1.18: Elephant-foot buckling (broad tanks)	35
Figure 1.19: Tanks Burn Down	35
Figure 1.20: Tank Farm on Fire	35
Figure 1.21: Mechanical Seal	37
Figure 1.22: Liquid-filled fabric seal	37
Figure 1.23: Lateral Deflection of Supporting Leg	39

Figure 1.24: Articulated Piping System	40
Figure 1.25: Flexible Steel Pipe System Inside the Tank	41
Figure 1.26: Articulated drain pipe system installed inside the tank	42
Figure 1.27: Flexible Swing Joint	42
Figure 1.28: Bleeder vents	43
Figure 1.29: Foam Fire Fighting System	45
Figure 2.1: Longitudinal forces acting on thin cylinder under internal Pressure	47
Figure 2.2: Circumferential forces acting on thin cylinder under internal Pressure	48
Figure 2.3: Circumferential Stress Thickness equation to 1-Foot method Equation	50
Figure 2.4: Diagrammatic sketch of shell wall with design thickness	51
Figure 2.5: Typical stiffener ring section for ring shell	52
Figure 2.6: Fabricated Tee Girder for Top Wind Girder	54
Figure 2.7: Height of transform shell	56
Figure 2.8: Fabricated Tee Girder for Intermediate Wind Girder	57
Figure 2.9: Overturning check on tank due to wind load	58
Figure 2.10: Summary Result for Overturning Stability against wind load	59
Figure 2.11: Seismic Diagram for a Floating Roof Tank	60
Figure 2.12: Design Response Spectral for Ground-Supported Liquid Storage Tanks	65
Figure 2.13: Sloshing Period Coefficient, $K_s$	66
Figure 2.14: Response Spectrum Curve	69
Figure 2.15: Effective weight of Liquid ratio	70
Figure 2.16: Center of Action for Effective Forces	72

Figure 2.17: Seismic Moment and Force Diagram	73
Figure 2.18: Annular Plate Requirement	76
Figure 2.19: Sloshing Wave of Liquid Inside Tank	78
Figure 3.1: Single deck Floating roof	80
Figure 3.2: Center deck and 2 adjacent compartments puncture	81
Figure 3.3: Minimum Requirement for Single Deck Pontoon Floating Roof	82
Figure 3.4: Case 1 – Dead Load Only	83
Figure 3.5: Case 2 – Dead Load + 10” Rain Accumulation	83
Figure 3.6: (a) Deck Deflection in Case 1	84
Figure 3.6: (b) Deck Deflection in Case 2	84
Figure 3.7: Radial Forces Acting on Pontoon Inner Rim	87
Figure 3.8: Sectional Detail of Pontoon	88
Figure 3.9: Standard Fitting and Accessories for Single Deck Roof	90
Figure 3.10: Pantograph Hanger	92
Figure 3.11: Scissor Hanger	92
Figure 3.12: Completed Assembled Pantograph	92
Figure 3.13: End Section Pantograph	92
Figure 3.14: Foam-Filled Seal	93
Figure 3.15: Liquid-Filled Seal	93
Figure 3.16: Secondary Seal	94
Figure 3.17: Number and Location of Support Legs	97
Figure 3.18: (a) Operating of Bleeder Vent during In-Breathing (Starting)	99
Figure 3.18: (b) Operating of Bleeder Vent during In-Breathing (Finishing)	99
Figure 3.19: (a) Operating of Bleeder Vent during Out-Breathing (Starting )	100
Figure 3.19: (b) Operating of Bleeder Vent during Out-Breathing (Finishing)	100

Figure 3.20: (a) Roof Drain with Roof Rise	104
Figure 3.20: (b) Roof Drain with Roof Fall	104
Figure 3.21: Articulated Drain Pipe System	105
Figure 3.22: (a) Typical Swing Joint in Articulated Drain Pipe System	106
Figure 3.22: (b) Swing Joint Assembly	106
Figure 3.23: Flexible Drain Pipe System	107
Figure 3.24: (a) Inner Section of COFLEXIP Pipe	108
Figure 3.24: (b) COFLEXIP Pipe of Different Size	108
Figure 3.25: End fitting of COFLEXIP Pipe	108
Figure 3.26: Flexible Drain Pipe System Installed in Different Tank	109
Figure 3.27: Sketch of Rolling Ladder and Gauger Platform in a Floating Roof Tank	112
Figure 3.28: Rolling Ladder and Gauger Platform Installed in a Floating Roof Tank	113
Figure 3.29: General Arrangement of the Multiple Foam Chamber on the Floating Roof Tank	114
Figure 3.30: (a) Fire Protection for Floating Roof Tank	115
Figure 3.30: (b) Foam Chamber	115
Figure 3.31: Typical Foam Dam	115
Figure 4.1: (a) Progressive Assembly & Welding and Complete Assembly Followed by Welding of Horizontal Seam Method for Welded Vertical Tank	116
Figure 4.1: (b) Jacking-Up and Flotation Method for Welded Vertical Tank	117
Figure 4.2: Tank Foundation with anchor bolt installed	118
Figure 4.3 Bottom Plate Layout	119
Figure 4.4: Bottom Plate Laid on Foundation	120

Figure 4.5: Typical Cross Joint in Three Plate Lap	120
Figure 4.6: Welding Detail for Bottom Plate	121
Figure 4.7: Completed Erection of First Shell Course	122
Figure 4.8: (a) Erection of Upper Shell Course – Inside Tank	122
Figure 4.8: (b) Erection of Upper Shell Course – Outside Tank	122
Figure 4.9: Vacuum Box and Pump	124
Figure 5.1: Maximum Allowable Sag	128
Figure 5.2: Maximum Tolerances for Out-of Verticality of the Tank Shell	129
Figure 5.3: Alignment of Shell Plate for Welding	130

## LIST OF FIGURE

## PAGE

Table 1.1: Process Design Data	17
Table 1.2: Nozzle Data	18
Table 1.3: Corrosion Rate Sensitively Result for 50% Summer and 50% Winter Condition	21
Table 1.4: Stress table for SA 516 Gr 65N	23
Table 1.5: Material Specifications for Stabilised Condensate Tank	24
Table 1.6: Material Selection Guide	24
Table 1.7: Bake Bean Can and Storage Tank Comparison Table	25
Table 1.8 (a): Fitting Requirements on Tank Shell	36
Table 1.8 (b): Fitting Requirement on Floating Roof	36
Table 2.1: Shell wall Design Thickness Summary	50
Table 2.2: Value of $F_a$ as a Function of Site Class	67
Table 2.3: Value of $F_v$ as a Function of Site Class	67
Table 2.4: Response Modification Factors for ASD Methods	68
Table 2.5: Summary of Design Parameter	68
Table 2.6: Anchorage Ratio Criteria	74
Table 3.1: Summary Result for Maximum Deflection and Stresses in Center Deck	86
Table 3.2: Summary Result for Pontoon Ring Stability	89
Table 3.3: Common Material for Select Product	95
Table 3.4: Properties of Common Seal Material	96
Table 3.5: Summary Result for Roof Support Legs	98
Table 3.6: Equivalent Pipe Length Chart	111

# CHAPTER 1: INTRODUCTION

## 1.1 Rationale

Floating roof tank is not a new technology or equipments and it had been widely used over the world in many industries. Storage tanks are designed, fabricated and tested to code and standard. There are a variety of codes and standards stating the similar common minimum requirements and some additional requirements from company standards or specifications.

Engineer or tank designer who do the preliminary and detail design are normally not familiar or not exposed to the actual site condition. Their designs are basically based on the code and standard requirements and basic theory from reference book. Some would only rely on the commercial software for the basic design, they have limited knowledge on the actual tank operation which limit them on cost effectiveness and even safety detail design, particularly on the floating roof tank.

There is limited procedure and rules in design the floating roof. These had resulted lots of floating roof failure in the industry. Hence industry, tank owner and also the tank designer or engineer need to have a simple rules and formula to ensure the floating roof is adequately designed and strong enough for the various loading during operation.

Beside of the procedures and rules, understanding of how the stresses behave in the tank material is essential for a complete safe design.

Floating roof tanks are usually built in a gigantic size and this would involve various disciplines such as civil, chemical, mechanical, fire safety, construction, inspection, commissioning and operation.

The work scope of each disciplines would have a direct effect on the tank design, one example is the tank foundation which is designed by the civil staff. The foundations are to be designed to withstand the load of the tank with its content. Improper design would result in foundation sagging or excessive soil settlement which in turn induces extra stresses to bottom of tank and tank shell.

Hence it is essential for the engineers or tank designer to know how and what effects each inter-discipline's design would have on one's tank that affected the tank integrity, and taking all these consideration into his design.

## **1.2 Research Goal**

### **1.2.1 Project Aims**

The aim of this project is to develop basic rules and procedures, highlighting the concerns in designing, construction and operation of a floating roof.

### **1.2.2 Project Objective**

The main objective of this project is "HOW TO DESIGN A NEW FLOATING ROOF TANK".

Taking an existing Oil Development Project with it's readily available information as a base, to design the tank, and identify the problematic and lesson learnt throughout the project.

## **1.3 Research Methodology**

### **1.3.1 Literature Review**

Literature review is conducted to study the basic design and requirement of the floating roof storage tank in the storage tank design code (API 650 – Welded Steel Tanks for Oil Storage).

Further studies on the tank design were made from other reference book, company standard specification and information from different disciplines.

### **1.3.2 Case Study**

Case studies on the previous project for the lesson learnt will be carried out.

### **1.3.3 Product Enquiries**

Research and study the role and application of the tank fittings and accessories by searching information and sending technical enquiries to the product supplier, attending the technical presentation conducted by the product supplier will be carried out.

### **1.3.4 Design Approach**

Upon completion of the literature review, design approach is then developed. The storage tank design consists of two major designs, that is (1) the shell design analysis and (2) the floating roof design.

In the shell design analysis, shell stress design will be performed taking into consideration of all the considerably loading including hydrostatic pressure, wind loading and seismic loading.

In the roof design, it consists of two sections, that is (1) roof stress design and the (2) roof fitting and accessories design.

Design calculation sheet using excel will be establish in the project.

Evaluation of the different type of roof fitting from different supplier with be carried out and selection of the fitting base the evaluation result.

### **1.3.5 Consequential effect of the design failure**

The relative importance of each fittings and accessories will be defined as well as the consequential effects it would have in case of malfunction.

### **1.3.6 Special Design and Construction**

Upon completion of the tank design, special consideration on the design and construction will be addressed base on the case study on the lesson learn and design process.

## **CHAPTER 2: LITERATURE REVIEW**

### **2.1 Introduction**

Storage tanks had been widely used in many industrial established particularly in the processing plant such as oil refinery and petrochemical industry. They are used to store a multitude of different products. They come in a range of sizes from small to truly gigantic, product stored range from raw material to finished products, from gases to liquids, solid and mixture thereof.

There are a wide variety of storage tanks, they can be constructed above ground, in ground and below ground. In shape, they can be in vertical cylindrical, horizontal cylindrical, spherical or rectangular form, but vertical cylindrical are the most usual used.

In a vertical cylindrical storage tank, it is further broken down into various types, including the open top tank, fixed roof tank, external floating roof and internal floating roof tank.

The type of storage tank used for specified product is principally determined by safety and environmental requirement. Operation cost and cost effectiveness are the main factors in selecting the type of storage tank.

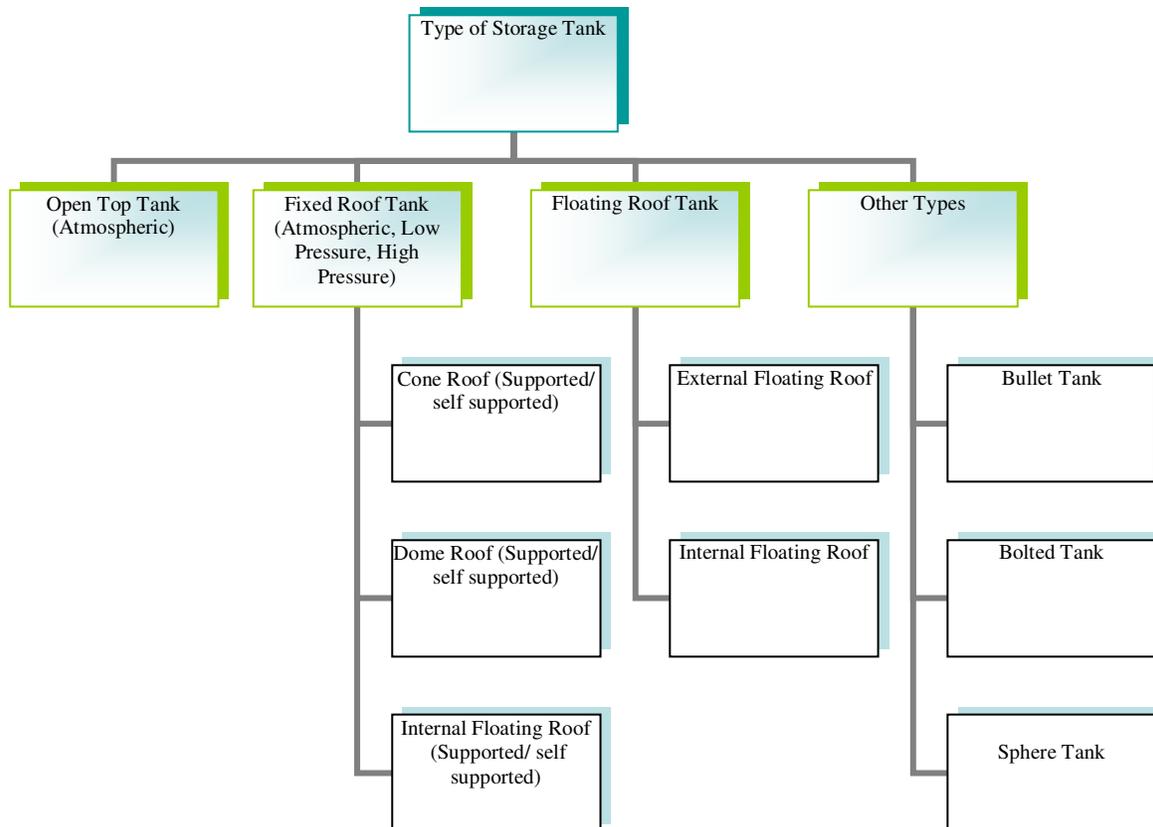
Design and safety concern has come to a great concern as reported case of fires and explosion for the storage tank has been increasing over the years and these accident cause injuries and fatalities. Spills and tank fires not only causing environment pollution, there would also be severe financial consequences and significant impact on the future business due to the industry reputation. Figure 1.1 shows the accident of the tanks that caught on fire and exploded. Lots of these accidents had occurred and they are likely to continue unless the lessons from the past are correctly learnt.



**Figure 1.1 Fire and explosion incidents in the tanks**

## 2.2 Types of Storage Tank

Figure 1.2 illustrates various types of storage tank that are commonly used in the industry today.



**Figure 1.2 Types of storage tank**

### 2.2.1 Open Top Tanks

This type of tank has no roof. They shall not be used for petroleum product but may be used for fire water/ cooling water. The product is open to the atmosphere; hence it is an atmospheric tank.

## 2.2.2 Fixed Roof Tanks

Fixed Roof Tanks can be divided into cone roof and dome roof types. They can be self supported or rafter/ trusses supported depending on the size.

Fixed Roof are designed as

- Atmospheric tank (free vent)
- Low pressure tanks (approx. 20 mbar of internal pressure)
- High pressure tanks (approx. 56 mbar of internal pressure)

Figure 1.3 shows the three types of Fixed Roof Tanks.

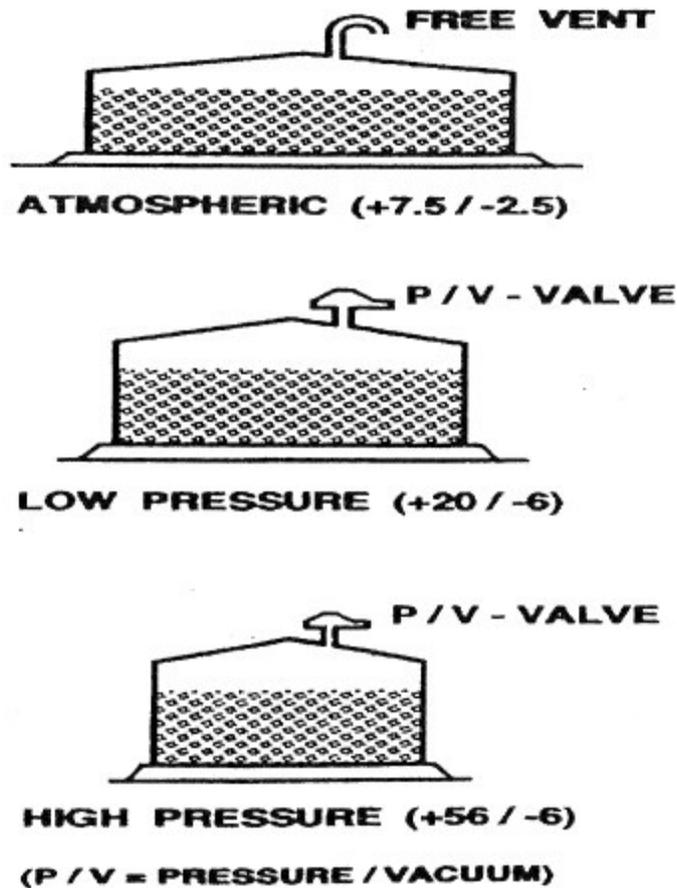


Figure 1.3 Types of Fixed Roof Tanks [EEMUA 2003, vol.1, p.11]

### 2.2.3 Floating Roof Tanks

Floating roof tanks is which the roof floats directly on top of the product.

There are 2 types of floating roof:

**Internal floating roof** is where the roof floats on the product in a fixed roof tank.

**External Floating roof** is where the roof floats on the product in an open tank and the roof is open to atmosphere.

Types of external floating roof consist of:

- Single Deck Pontoon type ( Figure 1.4)
- Double deck ( Figure 1.5)
- Special buoy and radially reinforced roofs

Floating roof tank will be further discussed in details in later chapter.

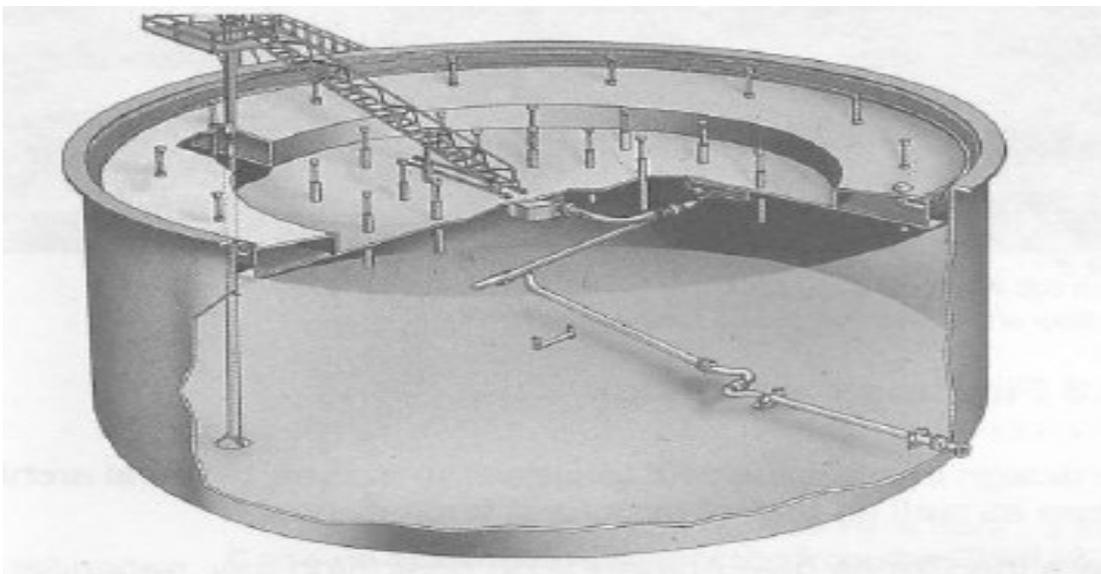
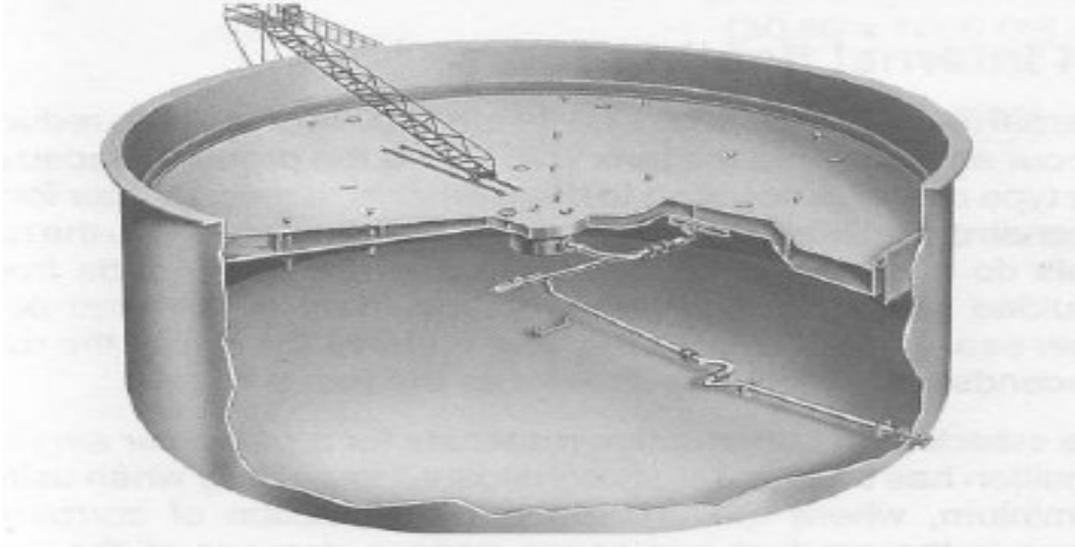


Figure 1.4 Single Deck Pontoon Type Floating Roof [Bob. L & Bob. G, n.d, p.155]



**Figure 1.5 Double Deck Type Floating Roof [Bob. L & Bob. G, n.d, p.155]**

## **2.3 Design Codes and Standards**

The design and construction of the storage tanks are bounded and regulated by various codes and standards. List a few here, they are:

- American Standards API 650 (Welded Steel Tanks for Oil Storage)
- British Standards BS 2654 (Manufacture of Vertical Storage Tanks with Butt-welded Shells for the Petroleum Industry)
- The European Standards
  - German Code Din 4119 – Part 1 and 2 (Above Ground Cylindrical Flat Bottomed Storage Tanks of Metallic Materials)
  - The French Code, Codres – (Code Francais de construction des reservoirs cylindriques verticaux en acier U.C.S.I.P. et S.N.C.T.)
- The EEMUA Standards (The Engineering Equipments and Materials Users Association)
- Company standards such as shell (DEP) and Petronas (PTS)

## **2.4 Floating Roof Tanks**

### **2.4.1 History and Introduction**

Floating roof tank was developed shortly after World War I by Chicago Bridge & Iron Company (CB & I). Evaporation of the product in fixed roof caused a great loss of money; this led to research to develop a roof that can float directly on the surface of product, reducing the evaporation losses.

### **2.4.2 Principles of the Floating Roof**

The floating roof is a circular steel structure provided with a built-in buoyancy which allows it to sit/ float on top of the liquid product in a close or open top tank.

The overall diameter of the roof is normally 400 mm smaller than the inside diameter of the tank, which has about 200 mm gap on each side between the roof and the inside tank wall. This is due to the limitation on the accuracy of dimension during construction for the large diameter tank. The gaps allow the floating roof to rise and fall without binding on the tank wall.

To protect the product inside the tank from evaporation to the atmosphere and contamination from the rain water through the gaps between the outer rim of the floating roof and the tank wall, the gaps will be closed or sealed up by means of a flexible sealing system.

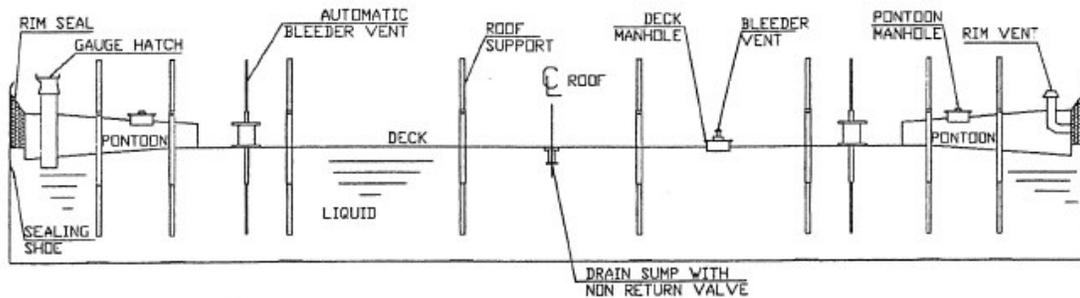
Due to environmental issues, selection of the roof seal is one of the major concerns in the floating roof tank design.

In single deck roof which shown in Figure 1.6, is also called pontoon roof, the buoyancy is derived in the pontoon, an annular circular pontoon radially divided into liquid tight compartments.

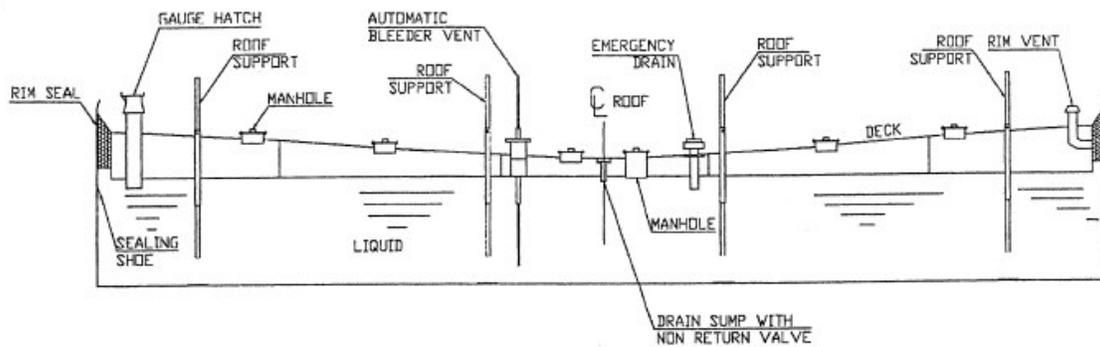
The center deck which is formed by membrane of thin steel plates are lap welded together and connected to the inner rim of the pontoons.

Double deck roof (Figure 1.7) consists of upper and lower steel membranes separated by a series of circumferential bulkhead which is subdivided by radial bulkhead. The outer ring of the compartments is the main liquid tight buoyancy for the roof.

Double deck roof is much heavier than single deck one, hence it is more rigid. The air gap between the upper and bottom plates of the deck has insulation effect which helps against the solar heat reaching the product during the hot climate and preventing heat loss of the product during cold climate.



**Figure 1.6 Single Deck Floating Roof Tank [EEMUA 2003, vol.1, p.15]**



**Figure 1.7 Double Deck Floating Roof Tank [EEMUA 2003, vol.1, p.15]**

### **2.4.3 Advantages of the floating roof storage tank**

As the roof floats directly on the product, there is no vapour space and thus eliminating any possibility of flammable atmosphere. It reduces evaporation losses and hence reduction in air pollution. Vapour emission is only possible from the rim seal area and this would mainly depend on the type of seal selected and used.

Despite of the advantages of the floating roof, to design and construct a floating roof tank will be much more complicated and costly than the fixed ones. In term of tank stability and design integrity, floating roof tank is never better than the fixed roof tank as there are still many unknown parameters and factors in designing the floating roof.

## **2.5 Design Data Overview**

Site geometric data are:

The plant is located in Kiyanlı, Balkanabad District in Turkmenistan located onshore by Caspian Sea.

The climate is sub tropical with hot dry summer and cold wet winter. The climate condition is as follow:

a. Temperature:

- Ambient: Mean annual = 14.6°C  
Extreme low = -17.0°C (January 1969)  
Extreme high = +44.0°C (July 1983)
- Design temperature change = +30°C

b. Rainfall Intensity:

- Maximum daily rainfall (4<sup>th</sup> May 1972) : 68 mm
- Maximum rain density once in 100 years : 0.69 mm/min
- Maximum rain density once in 50 years : 0.59 mm/min
- Maximum rain density once in 2 years : 0.3 mm/min

c. Humidity:

- Summer : 50% at 34°C
- Winter : 74% at 7°C

d. Wind Speed at 10 m above Ground level:

	<b>Operating</b>	<b>1 yr</b>	<b>10 yr</b>	<b>50 yr</b>	<b>100 yr</b>
<b>1 hour mean m/s</b>	12	17	21	24	25
<b>10 minutes mean m/s</b>	13	19	23	26	27
<b>1 minute mean m/s</b>	14	21	25	28	29
<b>3 second gust m/s</b>	15	23	27	31	32

e. Earthquake (MSK 64):

- Earth Tremor Intensity (severe damage to building) : 9
- Index of Earth Tremor Category (once in 1000 years) : 2

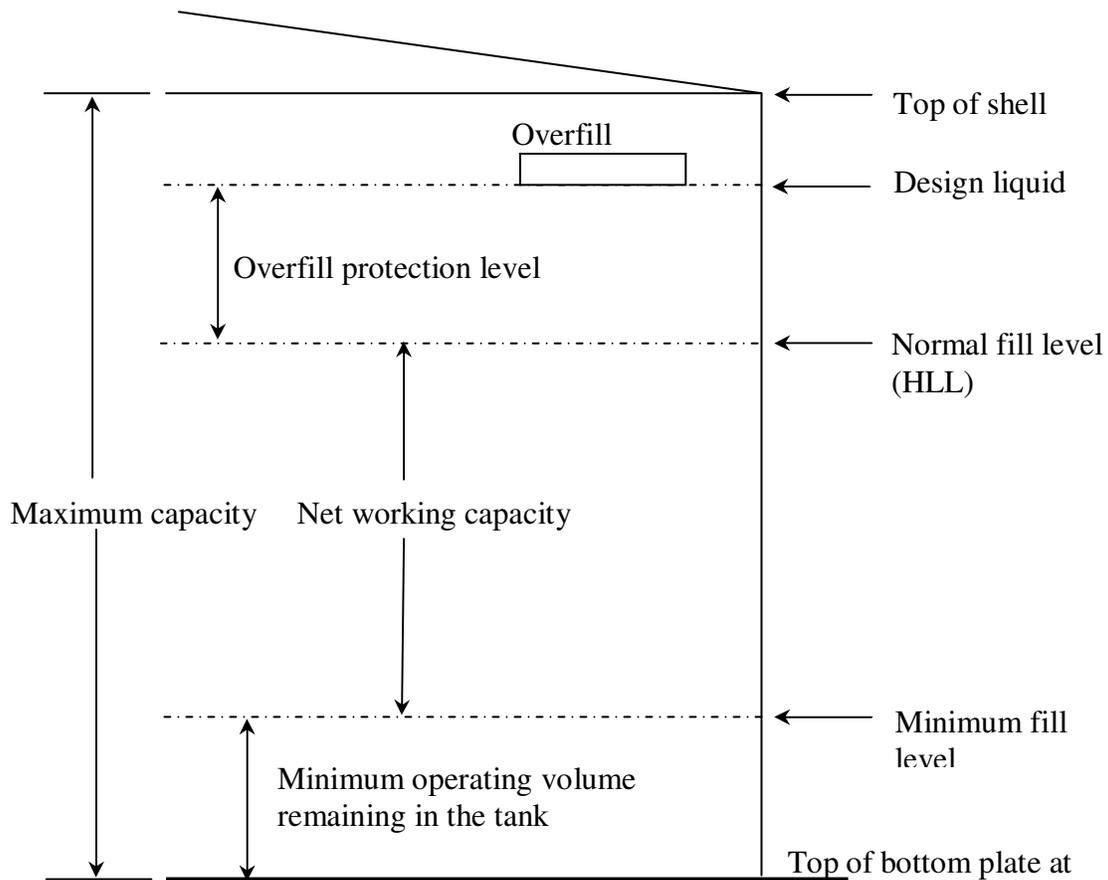
*Equivalent to Uniform Building Code (UBC) Zone 4*

f. Design Snow Loading : 56 kg/m<sup>2</sup>

## 2.6 Process Description and Requirements

Capacity determination is the one of the first steps in designing the tank. Only after the capacity is known, the tank can be sized up.

The definition of the maximum capacity can be explained easily in Figure 1.8.



**Figure 1.8 Storage Tank Capacities and Levels**

The maximum or total capacity is the sum of the inactive capacity (minimum operating volume remaining volume in tank), actual or net working capacity and the overfill protecting capacity.

The net working capacity is the volume of available product under normal operating conditions, which is between the low liquid level (LLL) and the high liquid level (HLL).

The storage tank capacity is sized in accordance with 85, 000 barrel tanker and 3 days of unavailability of the off loading system at production rate 51 000 barrels per day.

## **2.7 Process Description and Design Considerations**

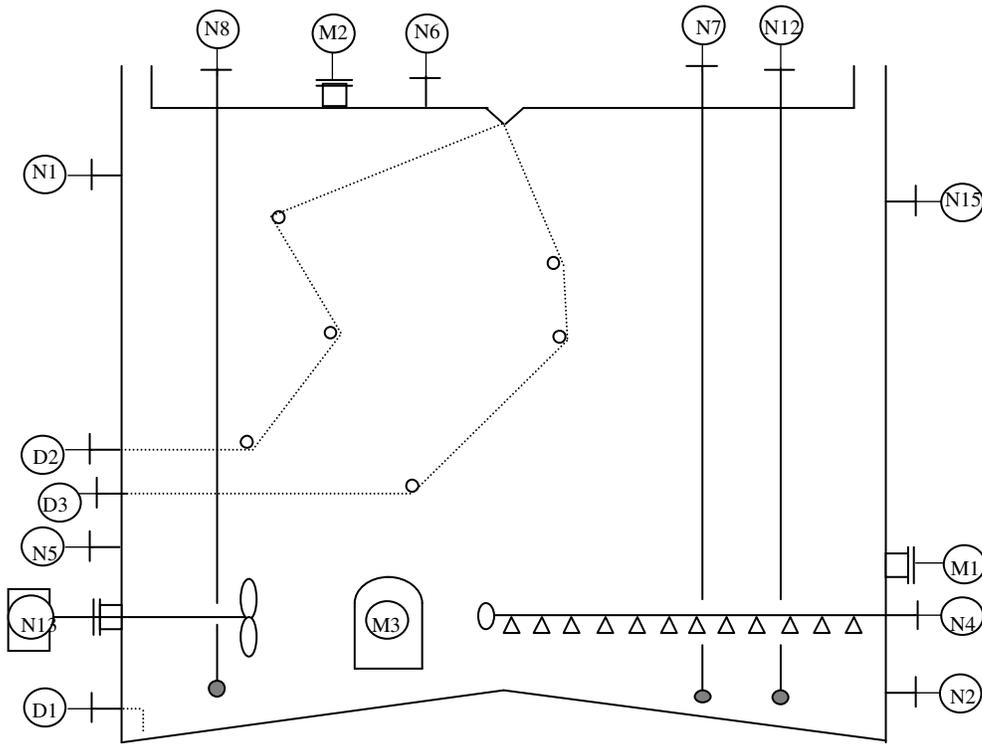
This storage tank is designed to store the stabilised condensate which runs down from the condensate stabiliser column. The stabilised condensate processed in the stabilised system is pumped to Stabilised Condensate Tank prior to export via underwater pipeline to the Single Buoying Mooring for ship loading.

Due to the waxy nature of the condensate, the liquid is heated above the wax dissolution temperature (WDT) of 39°C to prevent wax precipitation and formation in the pipeline.

The condensate in the tank is circulated in an external heating circuit to maintain the operating temperature at 44°C.

The stabilised condensate storage tanks are also equipped with motorized side entry tank stirrers to blend the storage fluid to ensure uniform temperature distribution in the tanks. It helps to prevent localized cooling that will result in wax formation in the storage tank.

The schematic sketch of the stabilized condensate tank is shown in Figure 1.9 with the process design data and nozzle data in Tables 1.1 and 1.2 respectively.



**Figure 1.9 Schematic Sketch of the Stabilised Condensate Tank**

<b>Service</b>	<b>Stabilised Condensate Tank</b>	
Tank Type	Floating Roof	
Number Required	Two ( 2 )	
Working Capacity	20000	m <sup>3</sup>
Nominal Capacity	24278	m <sup>3</sup>
Diameter	39000	mm
Height	20700	mm
Design Pressure	Atmospheric	
Operating Temperature	44	°C
Design Temperature	70 / -17	°C
Specific Gravity at 15°C/ at T	0.7903/ 0.7804	
Normal Filling Flow Rate	338	m <sup>3</sup> /h
Maximum Filling Flow Rate	427	m <sup>3</sup> /h
Normal Draw-Off Flow Rate	660	m <sup>3</sup> /h
Gauging Hole	No	
Level Indicator/ Alarms	Yes	
Mixing Propeller	Yes	
Manhole/ Inspection Hatches	Yes	
Insulation	Yes (Shell and roof)	

**Table 1.1 Process Design Data**

Nozzle Data				
Tag	No. Req.	Size (DN)	Service	Remark
N1	1	250	Inlet	
N2	1	450	Pump Suction	
N4	1	200	Recirculation Inlet	
N5	1	300	Recirculation Inlet	
N6	1	<i>Note 2</i>	Auto Bleeder Vent	
N7	1	100	Level Indicator	
N8	1	200	Level Transmitter	
N12	1	50	Temperature Transmitter	
N13	3	600	Mixing Propeller	<i>Note 3</i>
N15	1	200	Minimum Flow	
D1	1	100	Drain	
D2/ D3	2	100	Roof Drain	<i>Note 1</i>
M1	1	600	Shell Manway	
M2	1	600	Roof Manway	
M3	1	1200 x 1200	Clean Out Door	<i>Note 4</i>

**Table 1.2 Nozzle Data**

The following points are to be included in design considerations:

- 1) Quantity and size of the roof drain shall be designed and size up accordance to the rainfall intensity.
- 2) Auto Bleeder vent is required as per API 650 code, quantity and size to be designed accordance to the maximum filling and draw off rate [API650, 2007].
- 3) Tanks are fixed with 3 mixing propellers, they shall remain submerged below the low liquid level during operation.
- 4) Clean out door shall be suitable for wheel barrow access for facilitating sediment/ sludge cleaning process.
- 5) Tank bottom to be cone-up toward center.

## **2.8 Material Selection and Corrosion Assessment**

Material selection study was carried out by the material specialist to review the conceptual design basic of the plant and assess expected longevity of materials for various piping and equipment, he/she then proposes materials suitable for the required design life of 30 years. The approach of this material selection is to evaluate the internal corrosivity of the fluids with respect to utilisation of carbon steel.

Carbon Steel is considered as first choice, due to its lower cost, ready availability and well understood requirements to fabrication and testing. Material selection for the hydrocarbon system is based on detail evaluation of fluid properties, particularly using the carbon dioxide models.

### **2.8.1 CO<sub>2</sub> Corrosion**

Carbon dioxide dissolves in water and dissociates to form weak carbonic acid which causes corrosion on carbon steels. Higher partial pressures of CO<sub>2</sub> imply more dissolved CO<sub>2</sub> and hence higher corrosion rate. Higher temperatures and pressure increase the corrosion rate, but in certain conditions, about 70 to 80°C, a protective carbonate scale can form on the steel surface that reduces the corrosion rate, compared to lower temperatures where the scale does not form.

Corrosion resistant alloys (CRA) are used to avoid corrosion at high CO<sub>2</sub> contents, and in less corrosive condition and where required lifetime is limited, but it would be more economical to use carbon steel with a corrosion allowance and/or chemical inhibitor treatment. The presence of CO<sub>2</sub> infers that carbon steel will have finite life due to the wall thinning, a corrosion allowance is practical to accommodate up to 6mm.

Other concerns for the material selection are:

i) Material at minimum temperature

At low temperatures, ferritic steels (unalloyed and low alloy steels, and ferritic-austenitic duplex stainless steels), lose their ductility spontaneously as the materials are cooled, allowing any cracks and crack-like defects, that are harmless at normal operating temperatures, to propagate under load.

To have greater resistance to low temperature embrittlement, materials and welds are to be heat treated where applicable eg. normalised and post weld heat treated low alloy and carbon steel). For an even lower service temperature, fine grained materials are required, high nickel steels, or austenitic materials have to be used.

The seasonal changes in ambient temperatures require that low temperature properties of materials must be selected.

ii) Mercury

Stabilised condensate from Turkmenistan was measured to contain Hg 4µg/kg. [13]

Mercury (Hg) is a trace component of all fossil fuels. It is therefore present in liquid hydrocarbon and natural gas deposits, and may transfer into air, water and soil.

Materials unsuitable for hydrocarbon streams in presence of mercury due to liquid metal embrittlement, which will result in crack are:

- Aluminium and Aluminium Alloys
- Titanium and Titanium Alloys
- Copper and Copper Alloys
- Zinc and Zinc Alloys

Recommended materials are:

- Carbon steels and low alloy steels
- Stainless steels (Austenitic stainless steel, Duplex stainless steel)
- Nickel Alloys (Inconel 625, 825 and Monel)

### 2.8.2 Carbon Dioxide Corrosion Modeling

In the material selection study report, the design corrosion rate for carbon steel was calculated using the NORSOK “CO<sub>2</sub> Corrosion Rate Calculation Model” - M-506” [14]. This model is a development of the original work by De, Waard, Milliams and Lotz , and includes some effects due to the wall fluid shear stress.

The calculated results for the corrosion rate sensitivity for 50% summer and 50% winter condition is summarized in Table 1.3.

		mm/ year
Corrosion rate Case Sensitive (Summer)	Without Inhibitor	0.0033
	With Inhibitor	0.00033
Corrosion Allowance for 30yrs Design Life (50% Summer condition)	Without Inhibitor	0.0495
	With Inhibitor	0.00495
Corrosion rate Case Sensitive (Winter)	Without Inhibitor	0.0033
	With Inhibitor	0.00033
Corrosion Allowance for 30yrs Design Life (50% Winter condition)	Without Inhibitor	0.0495
	With Inhibitor	0.00495

**Table 1.3 Corrosion Rate Sensitively Result for 50% Summer and 50% Winter Condition**

The design life of 30 years is required and a typical 3 and 6mm corrosion allowance is used as the basic for the selection of carbon steel. For 30 years service, the maximum time-averaged corrosion rates that can be accommodated by a 3mm and 6mm corrosion allowance are 0.1 mm/years and 0.2 mm/year respectively. Therefore, based on the calculated result, low temperature carbon steel (LTCS) + 3 mm corrosion allowances + internal lining is recommended.

## **2.9 Mechanical Selection of Carbon Steel Grade**

Mechanical selection of material is based on their mechanical properties and their constructability. A 516 Gr 65N (ASTM low temperature carbon steel with minimum tensile of 65 ksi) is selected for its well known properties in low temperature. The material will be normalised.

Accordance to UCS-66, ASME VIII division 1 [2], A 516 Gr 65 without normalisation with fall under curve B and the material A 516 Gr 65N (Normalised) with fall under curve D (Figure 1.10).

From the impact test exemption curve in Figure 1.10 , it can be found that with the minimum design temperature of  $-17^{\circ}\text{C}$ , impact test will be required when the plate thickness exceed 15mm for materials in Curve B, whereas impact test is exempted up to thickness 58 mm for material in Curve D.

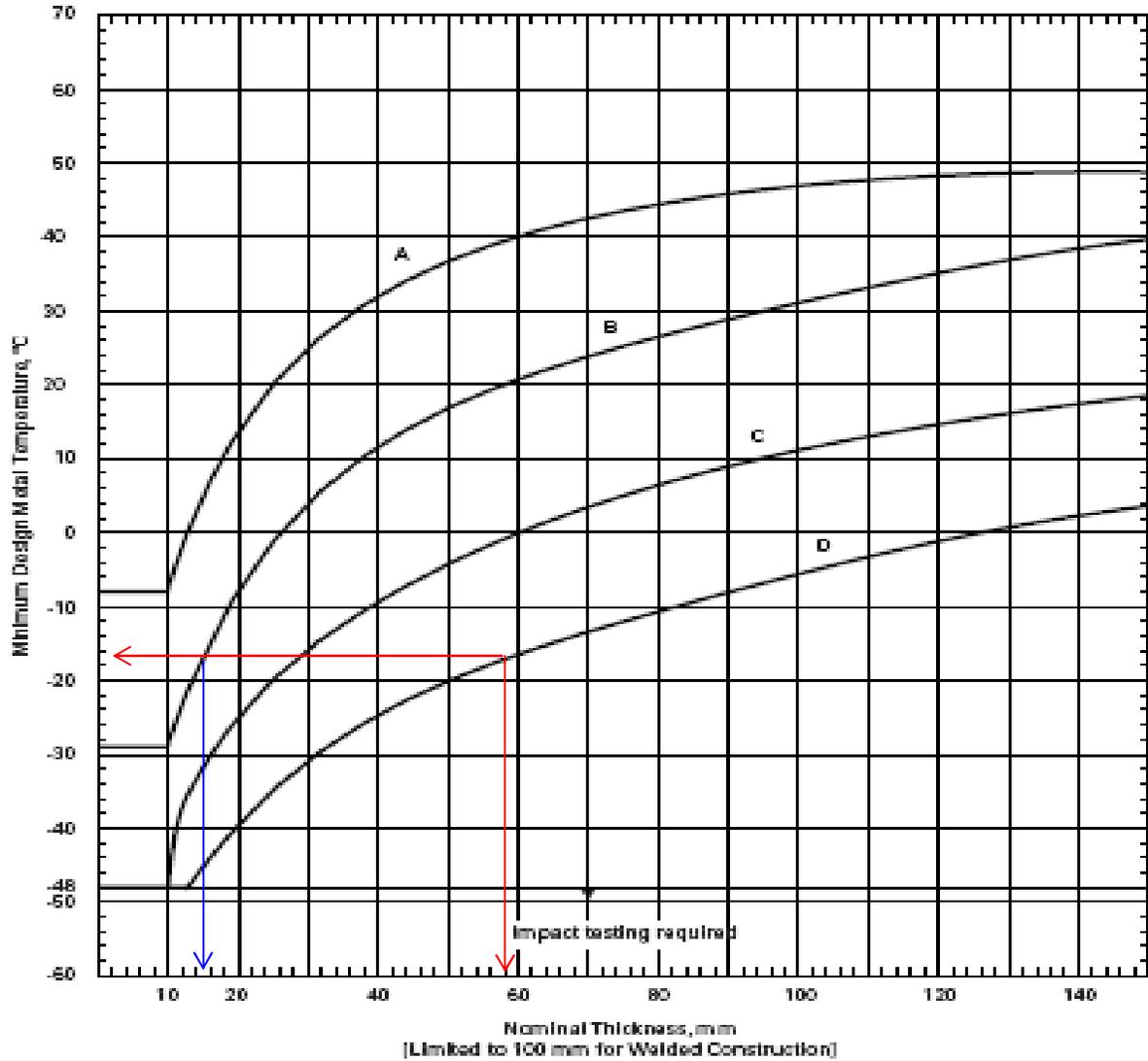


Figure 1.10 Impact Test Exemption Curve [ASME VIII, Div.1, 2007, UCS-66]

Mechanical properties for A 516 Gr 650N listed below are accordance to ASME II Part D – Material Property [3].

Minimum Tensile Strength	450 Mpa
Minimum Yield Strength	245 Mpa
Maximum Allowable Stress from -17°C to 100°C	128 Mpa

Table 1.4 Stress table for SA 516 Gr 65N

Tank Shell/ Bottom Plate	SA 516 Gr. 65N
Floating Roof	SA 516 Gr. 65N
Stiffener Ring	SA 516 Gr. 65N
Nozzle Neck Pipe (SMLS)	SA 333 Gr.6
Nozzle Flange/ Blind Flange	SA 350 Gr. LF2 Class 1
Nozzle Fitting	SA 420 Gr. WPL 6
Gasket	Flexible Graphite With Tanged Insert
Bolt & Nuts (External)	SA 320 – L7M/ SA 194 Gr. 2H (Fluorocarbon Coated)
Internal ( Bolting/ Piping/ Supports)	Stainless Steel SS 316L

**Table 1.5 Material Specifications for Stabilised Condensate Tank**

The material specification for the stabilised condensate tank is shown in Table 1.5. Table 1.6 illustrate the material selection guide, using design temperature to choose a readily available and cost effective material.

	Design Temperature, °F	Material	Plate	Pipe	Forgings	Fittings	Bolting
Cryogenic	-425 to -321	Stainless steel	SA-240-304, 304L, 347, 316, 316L	SA-312-304, 304L, 347, 316, 316L	SA-182-304, 304L, 347, 316, 316L	SA-403-304, 304L, 347, 316, 316L	SA-320-B8 with SA-194-8
	-320 to -151	9 nickel	SA-353	SA-333-8	SA-522-1	SA-420-WPL8	
Low temperature	-150 to -76	3½ nickel	SA-203-D	SA-333-3	SA-350-LF63	SA-420-WPL3	SA-320-L7 with SA-194-4
	-75 to -51	2½ nickel	SA-203-A		SA-350-LF2	SA-420-WPL6	
	-50 to -21	Carbon steel	SA-516-55, 60 to SA-20	SA-333-6			
	-20 to 4		SA-516-All	SA-333-1 or 6			
5 to 32	SA-285-C						
Intermediate	33 to 60 61 to 775		SA-516-All SA-515-All SA-455-II	SA-53-B SA-106-B	SA-105 SA-181-60,70	SA-234-WPB	SA-193-B7 with SA-194-2H
Elevated Temperature	776 to 875	C-½Mo	SA-204-B	SA-335-P1	SA-182-F1	SA-234-WP1	
	876 to 1000	1Cr-½Mo	SA-387-12-1	SA-335-P12	SA-182-F12	SA-234-WP12	
		1Cr-½Mo	SA-387-11-2	SA-335-P11	SA-182-F11	SA-234-WP11	
	1001 to 1100	2¼Cr-1Mo	SA-387-22-1	SA-335-P22	SA-182-F22	SA-234-WP22	with SA-193-B5 SA-194-3
	1101 to 1500	Stainless steel	SA-240-347H	SA-312-347H	SA-182-347H	SA-403-347H	SA-193-BB with SA-194-B
Incoloy		SB-424	SB-423	SB-425	SB-366		
Above 1500	Inconel	SB-443	SB-444	SB-446	SB-366		

**Table 1.6 Material Selection Guide [Moss, cited in Bednar 1991]**

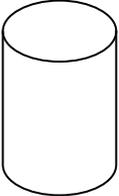
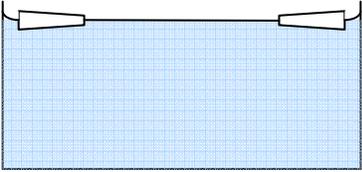
## 2.10 Mechanical Design

Stress design and analysis of the storage tank is the greatest concern to engineer as it provides the basic for the tank stability and integrity.

The basic stress analyses to be taken care in tank design are as follow:

- Tank shell wall due to internal and external loading
- Bottom plate/ Tank flooring
- Tank roof – In this case, floating roof

Storage tanks always look big and strong, and there are also often being referred as ‘tin can’. Some simple comparison in term of their sizes and strength is made here.

	Typical Bake Bean Can	Storage Tank
		
<b>Diameter, D</b>	75 mm	10, 000 mm
<b>Height, H</b>	105 mm	14, 000 mm
<b>Wall thickness, t</b>	0.15 mm	5 mm
<b>D/H ratio</b>	1 / 1.4	1 / 1.4
<b>t/D ratio</b>	0.002	0.0005

**Table 1.7 Bake Bean Can and Storage Tank Comparison Table**

From the Table 1.7, it can be seen found the tank ratio (t/D) is 4 times less than the typical bean can which show that how relatively flimsy the shell of the tank it would be if it is subjected to partial vacuum. Figure 1.11 shows an example of tank exploding due to vacuum loading.



**Figure 1.11 Tank Exploding [Bob.L & Bob.G, n.d, p.26]**

## **2.11 Tank Shell Design Method as Per API 650**

### **2.11.1 Calculation of thickness by 1-Foot Method**

The 1-foot method calculates the thickness required at design points 0.3 m (1 ft) above the bottom of each shell course.

The formula for the minimum required thickness is as followed:

For design shell thickness,

$$t_d = \frac{4.9(H - 0.3)G}{Sd} + C.A$$

For hydrostatic test shell thickness,

$$t_t = \frac{4.9(H - 0.3)}{S_t}$$

Where

- $t_d$  = Design shell thickness, in mm
- $t_t$  = Hydrostatic test shell thickness, in mm
- $D$  = Nominal Tank Diameter, in m
- $H$  = Design liquid level, in m
- $G$  = Design specific gravity of the liquid to be stored
- $C.A$  = Corrosion allowance, in mm
- $S_d$  = Allowable stress for the design condition, in MPa
- $S_t$  = Allowable stress for the hydrostatic test condition, in MPa

This method is shall not be used for tanks larger than 60 m in diameter.

### **2.11.2 Calculation of thickness by Variable-Design-Point Method**

Design using variable-design-point method gives shell thickness at design points that in the calculated stressed being relatively closed to the actual circumferential shell stress.

This method normally provides a reduction in shell-course thickness and total material weight, but more important is its potential to permit construction of large diameter tanks within the maximum plate thickness limitation.

This method may only be used when 1-foot method is not specified and when the following is true:

$$\frac{L}{H} \leq \frac{1000}{6}$$

### **2.11.3 Calculation of thickness by Elastic Analysis**

For tanks where  $L / H$  is greater than  $1000/6$ , the selection of shell thickness shall be based on an elastic analysis that shows the calculated circumferential shell stress to be below the allowable stress.

### **2.12 Mechanical Design Consideration**

The principal factors in determine the shell thickness is the loads, the primary loading to determine the basic shell thickness is as follow:

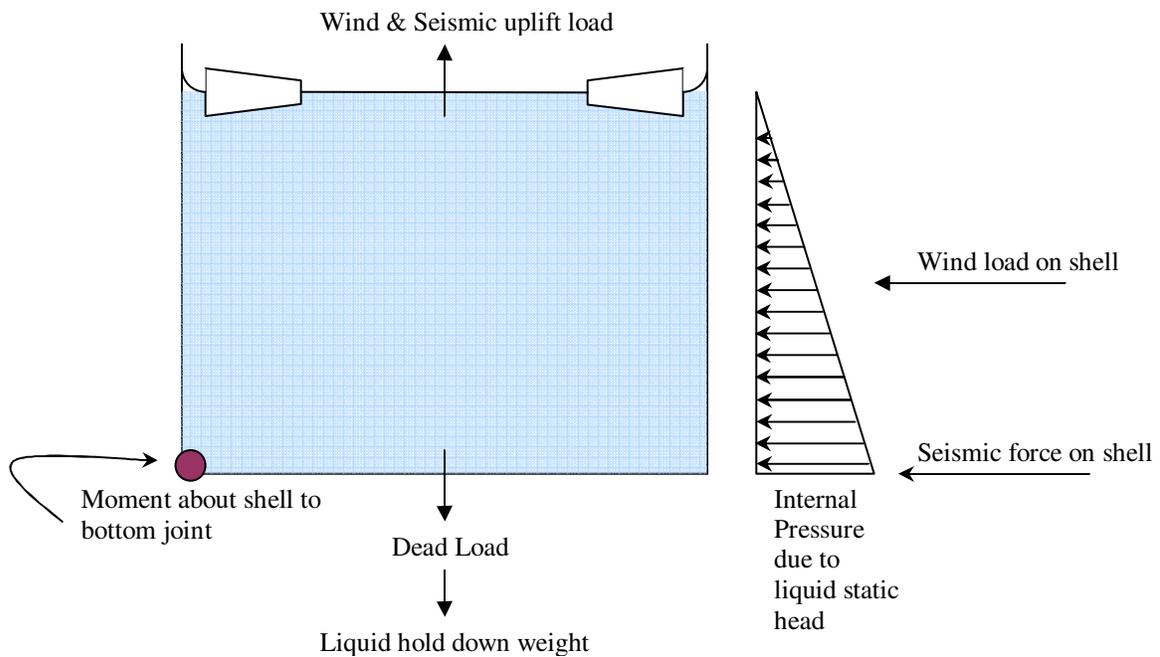
- The internal loading due to the head of liquid
- The pressure in the vapour space

(This factor is not applicable for floating roof tanks as the roof sit directly on the liquid, there is no vapour space.)

Other external loading shall be taken into consideration are:

- External pressure – Vacuum condition
- Wind loading
- Seismic Loading
- Localized loads resulting from nozzles, attachments, ladder/ stair and platform etc.

The primary loadings exerted to the tank shell are illustrated in Figure 1.12:



**Figure 1.12 Loading Diagram on a Tank Shell**

The internal pressure exerted on the tank shell is the product liquid head; the pressure is at the highest at the tank shell bottom and decreases linearly along its height. External loading of wind and seismic act on the tank shell and create an overturning moment about the shell to bottom joint, this results in the uplift reaction of the tank and affected the tank stability.

The various stresses to which the shell of a tank is subjected are

- **Hoop tension** which is caused by the head of product in the tank, together with any overpressure in the roof space of a fixed roof tank.
- **Axial compression** which comes from the tank self-weight, internal vacuum, wind and seismic loading acting on the shell which causes an overturning effect.
- **Vertical bending** due to the expansion of shell under normal service loading

## 2.13 Bottom Plate Design

API 650 has a very straight forward requirement on the bottom plate thickness and width requirement.

### 2.13.1 Vertical Bending of Shell

When the tank is filled with product, the shell will expand radially due to the elasticity of the shell plate material. This natural expansion is restricted at the point where the shell is welded to the bottom plate.

The shell-to-bottom joint is very rigid and it rotates as a unit when the tank is under hydrostatic load.

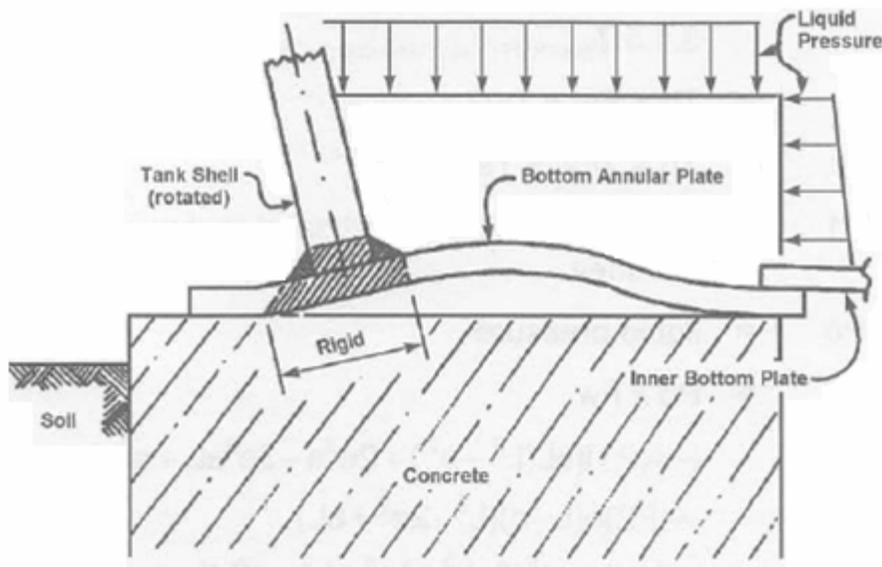


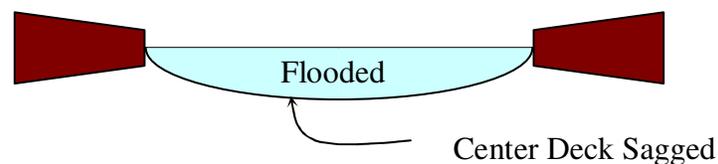
Figure 1.13 Rotation of the shell-to-bottom connection [Bob.L & Bob.G, n.d, p.47]

The shell tends to rotate in an outward direction about the rigid joint as depicted in Figure 1.13, the bottom plate will also rotate and cause it to lift off the foundation for a distance

inside the tank until the pressure of the product acting on the floor, balances the lifting effect.

This action causes high bending stresses in the bottom plate and the toe of the internal fillet weld. Due to the continual filling and emptying of the tank, the load is cyclic and this area is subject to low cycle fatigue.

## 2.14 Floating Roof Design



**Figure 1.14 Single Deck Roof Sagged with Flooding Rain Water**

In API 650 (2007), the external floating roof is covered in Appendix C, it gives guidance and provides minimum requirement on the external floating roof design. Similar minimum requirement were also provided in the BS 2654 where they both stated that the pontoon volume shall be designed to have sufficient buoyancy to remain afloat on the liquid with specific gravity of the lower of the product specific gravity or 0.7 with the primary drain inoperative for the following conditions:

- the deck plate and any two adjacent pontoon compartments punctured and flooded the single deck or double deck pontoon roof.
- Rainfall of 250 mm (10 in.) in 24 hour period over the entire horizontal roof area.

These two codes also provide some minimum requirements on the roof fittings and accessories to optimize the floating roof design ensuring the roof is functioning effectively.

Though the codes addressed the minimum requirement on the pontoon volume, there is no mention on the structural adequacy. There is no proper procedure or standard and firm rules stated in any code or engineering handbook in designing the floating roof, as in structural integrity and buoyancy stability. It is always left to the designer or manufacturer to develop their own approaches to meet the minimum requirement stated in API 650 (2007) or BS 2654. Industry or purchaser will have to rely on the tank and roof manufacturer for the safe design.

Hence, there is a wide variation in the floating roof design approach, wide variation in the durability and reliability of the tank, in which there are also many tank failure due to various design problem in each different approach.

If the floating roofs are inadequately designed or wrong approaches were applied to the design, the roof will fail, pontoon will buckled and damaged. The most common failure on the floating roof is the sinking of the floating roof. The floating roof overtopped by the liquid inside the tank and the roof sunk. To the worst case, the tank will catch fire due to the spark generated during the unstable movement of the roof.

## **2.15 Special Consideration**

### **2.15.1 Soil Settlement**

Tank foundation shall be carefully designed to ensure adequate for the tank support. Soil investigation and study are required to monitor the soil settlement. Soil settlement is a common problem in compressible soil, and it has consequential problems on the floating roof tank.

Storage tanks are relatively large but flimsy structures, having very flexible envelopes such that the tank shell and bottom will generally follow the settlements of the subsoil.

The dead weight of the tank structure is relatively small compared with the live load of the contents, hence at location where weak, compressible layers are present in the subsoil, excessive soil settlement may occur due to the weight of the tank and its liquid content. Excessive soil settlement can affect the integrity of tank shells and bottoms, and causes a dozens of consequential problems. Having reference from the EEMUA Publication No, 159 (2003) [5], a few of consequential problems are quoted below:

- Jamming of floating roof structure around guide pole
- Jamming of roof seals due to (progressively increasing) out-of-roundness of the tank shell
- Roof seals giving a gap as the result of out-of-roundness and/or tilting of the roof
- Loss of buoyancy of floating roofs due to liquid in pontoon
- Roof drain leaking or being blocked
- Derailing of rolling ladder on top of a floating roof
- Buckling of the supporting legs of a floating roof tank due to inadequate support, or vacuum conditions
- Wear and tear scratching shoe plates/ tank shell

### **2.15.2 Seismic Design For Floating Roof**

As mentioned earlier that the minimum requirement provided in the API 650 (2007) and BS 2654 addressed only the floating consideration. The floating roof was simplified and assumed as rigid body, dynamic of the flooding and sloshing of the product was not considered. The behavior of floating roofs under seismic condition is very less, and sloshing behavior during seismic is complicated. Industry and owner normally depend on the tank and roof manufacturer for safe design, however, most of the floating roof tanks built do not consider the seismic condition in their roof design as code never addresses it.

Tanks had suffered significant damage during past earthquakes, some history cases of tank failure due to the sloshing wave are:

- Hokkaido, Japan in 2003 [John, 2006]
  - Fully Involved Tank Fires
  - Fully Involved Due to Floating Roof Collapse from Sloshing waves
  - 50% due to Sloshing Wave
  
- Ismir, Turkey in 1998 [John, 2006]
  - 23 Major Tank Fires
  - 17 Due to Sloshing Wave
  - 50% Due to Sloshing Wave

## 2.16 Failure Modes Due To Seismic Effects On Floating Roof Tank

There are three cases of a few on the roof,

- Roof collapse or Sinking
- Overtop of floating roof by the liquid inside the tank (Figure 1.15)
- Pontoon Buckling (Figure 1.16)



**Figure 1.15 Floating roof overtopped [Praveen, 2006]**



**Figure 1.16 Pontoon buckling [Tetsuaya, 2007]**

There is one case on shell,

- Shell Buckling caused by combination of outward pressures generated by vertical motion and compressive stresses generated by horizontal motion



**Figure 1.18 Elephant-foot buckling (broad tanks)**  
[Praveen, 2006]



**Figure 1.17 Diamond buckling (slender tanks)**  
[Praveen, 2006]

And one case on Tank Farm/ Plant

- Tanks burn down, the tanks caught fire due to sparks generated by up-down movement of the roof against the guides



**Figure 1.19 Tanks Burn Down** [John, 2006]



**Figure 1.20 Tank Farm on Fire** [Praveen, 2006]

## 2.17 Fitting Design and Requirement

A complete set of fitting and accessories are required for the floating roof to operate properly. It is essential to understand the function of each accessories and the situation that could cause the accessories to malfunction.

There are minimum requirements outlined for the fitting in API 650 (2007), and Petronas Technical Specification (PTS) has specified a requirement on the minimum number of fitting to be installed on the floating roof tank. Tables 1.8 (a) and (b) below show the fitting requirement as per PTS in the tank shell and floating roof respectively.

Fitting Description	Minimum Number Required
Shell Manhole	2 nos. of DN 600
Shell Inlet Nozzles	As specified by process design
Shell Outlet Nozzles	
Product Drain Nozzle and piping	
Water Drain Nozzle and piping	
Drain Sump	
Earthing Bosses on shell	
Shell manhole for mixers	
Clean out door	
Spiral Staircase	One Set

**Table 1.8 (a) Fitting Requirements on Tank Shell [PTS, 1986]**

Fitting Description	Minimum Number Required
Roof Drain System	One set
Roof drain sump	One set
Roof earthing equipment	One set
Roof Seal Mechanism	As specified by process design
Roller Ladder	One set
Roof Manhole	As specified by process
Roof Compartment manhole	As specified by process
Emergency Drain	One set for double deck only
Rim Vent	As specified
Roof Vent (Pressure/ Vacuum)	As Specified by process design
Automatic Bleeder Vent	One set
Dip Hatch	One set
Guide Device	One
Roof Supporting Legs	One set

**Table 1.8 (b) Fitting Requirement on Floating Roof [PTS, 1986]**

## 2.18 Typical Fitting and Accessories For Floating Roof

### 2.18.1 Roof Seal System

As mentioned early in the principal of floating roof, roof seal is used to prevent the escape of vapour from the rim gap and to minimise the amount of rain water entering the product. The sealing system has to be flexible enough to allow for any irregularities on the construction of the roof and shell when the roof moves up and down and for any radial or lateral movement of the roof due to wind and seismic.

There are several types of roof sealing system which consists of primary seal and secondary seal. Primary seals may comprise metallic shoes having flexible seals with a weight or spring-operated pusher mechanism, or be non-metallic tube seal, a fabric seal.

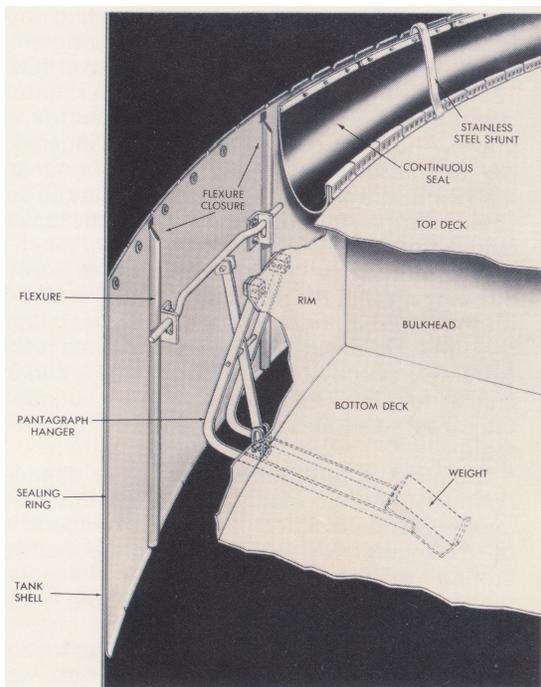


Figure 1.21 Mechanical Seal

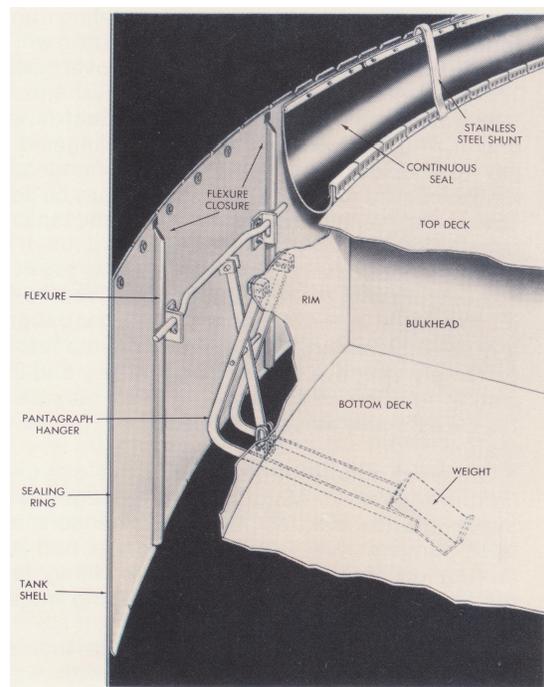


Figure 1.22 Liquid-filled fabric seal

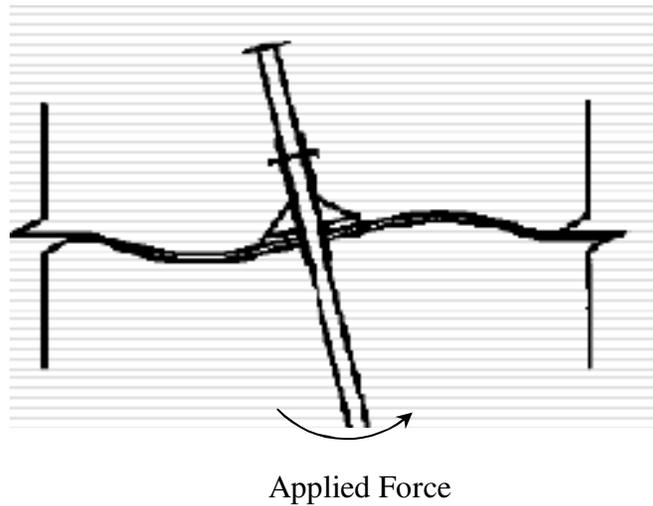
Primary seals were only used when floating roofs were first devised; secondary seals were the recent innovation to suit the new legislation in which the new limits of vapour emission was set. Secondary seals were mounted above the primary seal in which it can further reduce the vapour and odour losses from the floating roof tank.

The seals showing in Figure 1.21 and Figure 1.22 had been used for many years since floating roof were developed. The most recent innovation on the primary seal is the compression plate type and most of the tank owners are moving toward this new sealing system.

### **2.18.2 Support Leg**

Support leg is the supporting element for the floating roof when the tank is empty where the roof fall to its lowest position. The roof needed to be supported at a certain height above the floor not only that the roof will not foul with any internal accessories that installed at the lowest shell such as heating coil, mixing propeller, it also provide access room for maintenance personnel. As stated in API 650 (2007), the supporting legs can be either removable or non- removable type. The area of the tank floor in which the legs land shall be reinforced with a fully welded doubler plate which can distribute the leg loads into the floor plating.

More careful consideration will be required for the supporting requirement for the single deck pontoon roof as this type of roof is less rigid. Figure 1.23 shows that the deck is weak in bending and allows lateral deflection of the support leg.



**Figure 1.23 Lateral Deflection of Supporting Leg**

There is minimum requirement stated in API 650 (2007) where the legs and attachments shall be designed to the roof and a uniform live load of at least 1.2kPa. The legs thickness shall be Schedule 80 minimum and sleeves shall be schedule 40 minimum.

### **2.18.3 Roof Drain System**

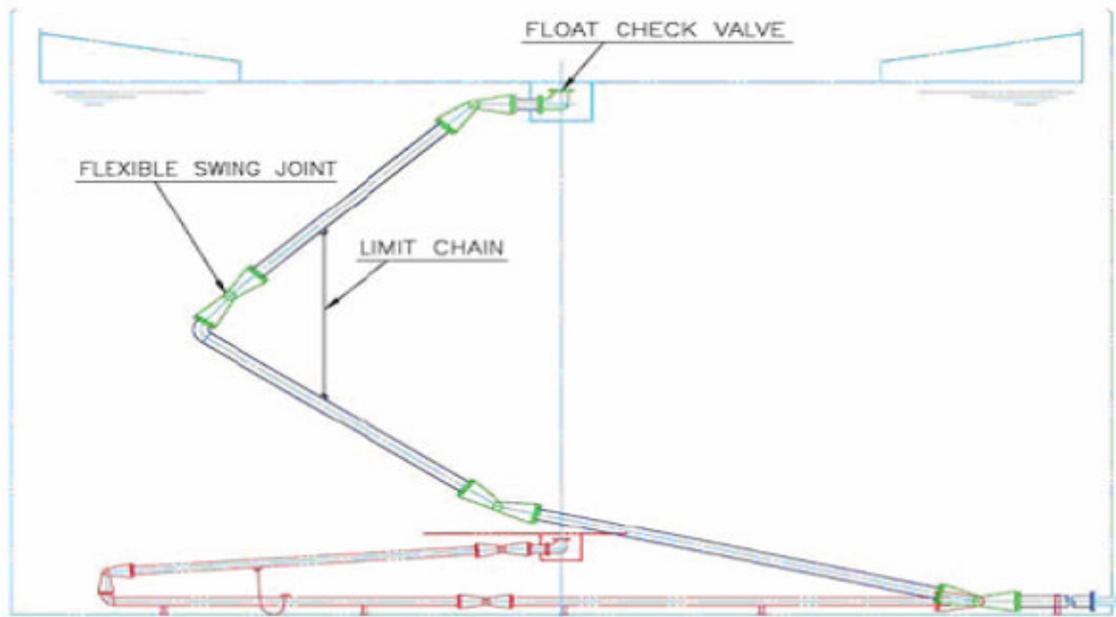
Roof drainage is one of the concerns in the roof designing; a reliable drainage system is indispensable for floating roof storage tanks. Improper roof drainage system would impair tank operation and threatens the safety of the stored product.

As addressed in API 650, the roof drains shall be sized and positioned to accommodate the rainfall rate while preventing the roof from accumulate a water level greater than design, without allowing the roof to tilt excessively or interfere with its operation.

The rain water which accumulates on the floating roof is drained to the sump which normally set in the low point of the deck. The sump will then be drained through a closed pipe work system inside the tank and drained out through the shell nozzle at the bottom side of the shell wall. A check valve is installed at the inlet of the drain.

The pipe work system which operates inside the tank has to be flexible to allow for the movement of the roof. The two most common used systems are the articulated piping system and the flexible pipe system.

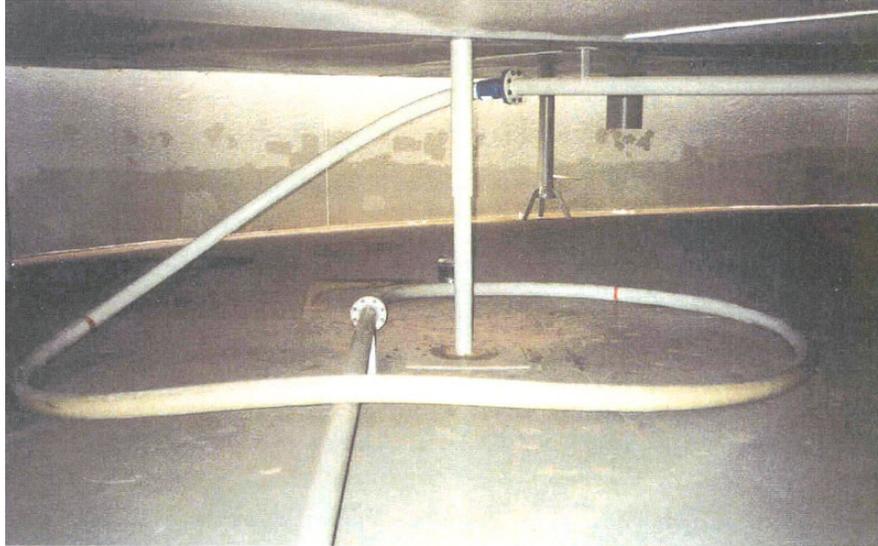
Articulated piping system uses solid steel pipe with a series of articulated knuckle joints or flexible swing joint. Figure 1.24 shows the articulated piping system in a floating tank.



**Figure 1.24 Articulated Piping System**

Flexible pipe system is installed in a single continuous length without ballasting or other devices. It maintains constant repeatable lay-down pattern on the tank floor, expanding and contracting with the rise and fall of the roof, not interfere with the equipment of accessories inside the tank.

Flexible pipe system consists of flexible rubber hose or steel pipe. However rubber is not recommended for oil industry. As stated in API 650 (2007), siphon type and non-armored hose-type are not acceptable as primary roof drain. Figure 1.25 shows photo of a flexible steel pipe system installed in a floating roof tank.



**Figure 1.25 Flexible Steel Pipe System Inside the Tank**

Emergency roof drain shall be installed, but only to double deck roof. Its purpose is to allow natural drainage of rainwater in case of malfunction of the primary drain. Emergency roof drains are prohibited by API 650 (2007) on the single deck pontoon roofs as the product level in the tank is always higher than the rainwater level in the centre deck, this would cause the product to discharge through the drain onto the roof rather than allow water to drain into the tank. It will also allow vapour to escape from the tank as it is an open drain. Even though emergency drain was addressed in the API 650 (2007) for double deck roof, some company had already banned the usage of the emergency drain.

Figure 1.26 and Figure 1.27 were taken in November 1993 at one of the refinery plant in Singapore where it showed an articulated drain system installed in the tank. This system had only in service for approximately 2.5 years; however considerable corrosion was observed on the end connector and the galvanized side plate.



**Figure 1.26 Articulated drain pipe system installed inside the tank**



**Figure 1.27 Flexible Swing Joint**

#### 2.18.4 Vent – Bleeder Vents

Automatic bleeder vents shall be furnished for venting the air to or from the underside of the deck when filling and emptying the tank. This is to prevent overstress of the roof deck or seal membrane. These vent only come to operate when the floating roof landed, and the tank is drained down or being filled.

Figure 1.28 shows the operation of the valve. The length of the push rod is designed in a way that as the tank is emptied, the rod touches the tank floor before the roof support leg landed and the will open automatically, freely venting the space beneath the deck. Similarly, when the tank is filling up, the valve closes after all the air beneath the deck has been expelled and the roof floats.

The number and size of the bleeder vent shall be sized accordance to the maximum filling and emptying rates.

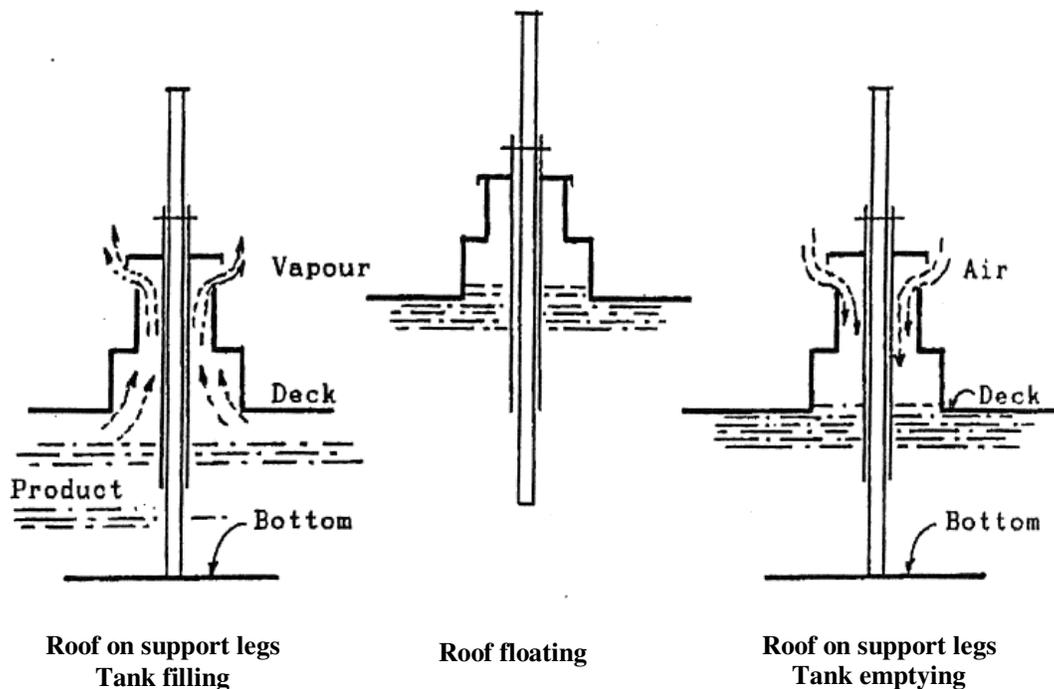


Figure 1.28 Bleeder vents [EEMUA 2003, vol.1, p.15]

### **2.18.5 Centering and Anti-Rotation Device**

Anti-rotation devices also called guide pole is required as stated in API 650 (2007) to maintain the roof in central position and prevent it from rotation. It shall be located near to the gauger platform and capable of resisting the lateral forces imposed by the roof ladder, unequal snow load and wind load.

### **2.18.6 Rolling Ladder and Gauger Platform**

Rolling ladder is the mean of access on to the floating roof. The upper end of the ladder is attached to the gauger platform and the lower end is provided with an axle with a wheel on side of ladder which runs on a steel track mounted on a runway structure supported off the roof. This is so that as the roof moves up and down, the ladder can slide along and take up vary angle as required. This is why the floating roof is always sized up in such a way that the tank diameter shall at least be equal to its height to enable the use of the rolling ladder for access to the roof.

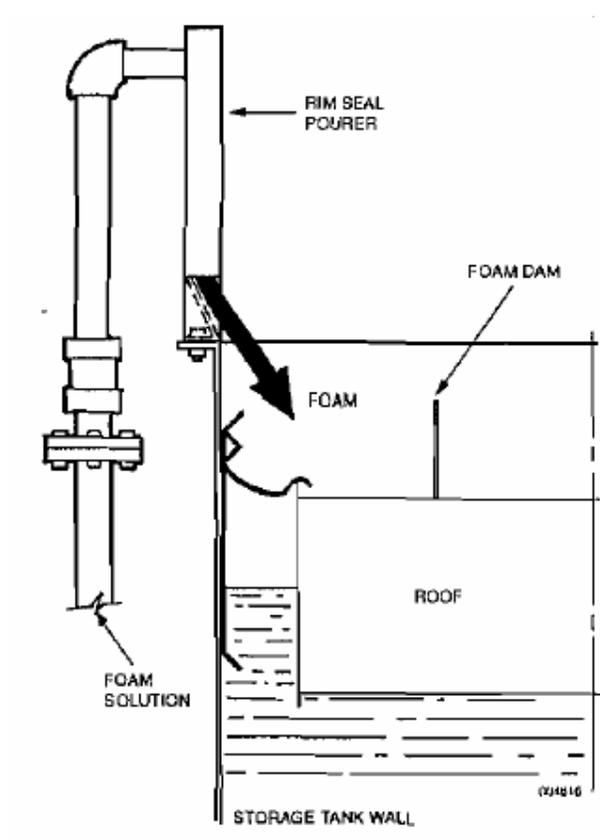
There will be a reaction at the lower end of the ladder causing a localized and eccentric load on the roof, this has to be taken into consideration while designing the roof. Gauger platform is a small access area overhangs the shell to allow the guide pole, and some other instrument to pass through providing access for the maintenance personnel.

### **2.19 Fire Fighting System and Foam Dam**

A fire detection system shall be installed when required, fires in floating roof tanks are usually in the area between the shell and the rim of the floating roof. The floating roof tanks shall be equipped with the fire fighting system, the foam system, which the system is designed to deliver a flame smothering expanded foam mixture into the tank rim space to extinguish the fire. A foam dam which consists of a short vertical plate is to welded to

the top pontoon plate at a short distance from the seal, with the height higher than the upper tip of the seal, to allow the whole seal area to flooded with the foam and extinguishes the fire effectively.

Figure 1.29 shows a typical arrangement of the foam system which it consists of a foam generated and pourer, installed around the tank periphery.



**Figure 1.29 Foam Fire Fighting System**

## CHAPTER 3: TANK DESIGN

### 3.1 Introduction

Storage tank design consists of 2 main sections – Shell Design and Roof Design. The shell design include the shell stress design which is to size up the shell wall thickness, top and intermediate stiffener ring, stability check against the wind and seismic load and sizing up the anchor bolt. The roof design will consist of roof stress design, and the roof accessories and fitting design.

### 3.2 Shell Design

The tank shell is designed accordance to the API 650 (2007) and the design considerations had been stated in the literature review under Chapter 2.12, Mechanical Design Consideration. It was also mentioned in the literature review that there are several methods stated in API 650 (2007) to determine the shell wall thickness. Based on the tank size of 39 m diameter, 1-Foot Method was the most appropriate method to be used. The 1-foot method calculates the thickness required at design points 0.3 m (1ft) above the bottom of each shell course.

The required minimum thickness of shell plates shall be the greater of the value computed as followed [API 650, 2007]:

Design shell thickness:

$$t_d = \frac{4.9D(H - 0.3).G}{S_d} + C.A$$

Hydrostatic test shell thickness:

$$t_t = \frac{4.9D(H - 0.3)}{S_t}$$

Where

$t_d$  = design shell thickness, mm

$t_t$  = hydrostatic test shell thickness, mm

$D$  = nominal tank diameter, m

$H$  = design liquid level, m

$G$  = design specific gravity of the liquid stored

$C.A$  = corrosion allowance, mm

$S_d$  = allowable stress for the design condition, MPa

$S_t$  = allowable stress for the hydrostatic test condition, MPa

The equation in the API 650 (2007) 1-Foot Method can be derived from the basic membrane theory, the two main stresses exerting on the cylindrical shell due to the internal pressure are longitudinal stress and circumferential stress. Let's look into each stress individually by analyzing the stresses in the thin-walled cylindrical shell which an internal pressure exerted on it.

### 3.2.1 Longitudinal Stress

Figure 2.1 show a thin walled cylindrical in which the longitudinal force  $F_L$  resulted from the internal pressure,  $P_i$ , acting on the thin cylinder of thickness  $t$ , length  $L$ , and diameter  $D$ .

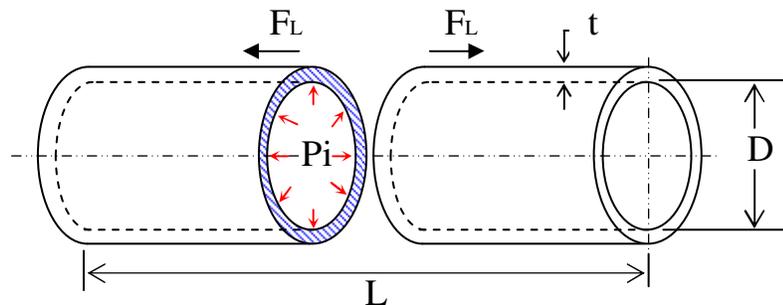


Figure 2.1 Longitudinal forces acting on thin cylinder under internal pressure

Longitudinal force,  $F_L = P_i \times \pi/4 \times D^2$

Area resisting  $F_L$ ,  $a = \pi \times D \times t$   
(Shade area)

$$\text{Longitudinal Stress, } S_L = \frac{\text{Longitudinal Force, } F_L}{\text{Resisting Area, } a}$$

$$S_L = \frac{P_i \cdot D}{4 \cdot t}$$

In term of thickness,

$$t_L = \frac{P_i \cdot D}{4 \cdot S_L}$$

We call this equation as Longitudinal Stress Thickness Equation.

### 3.2.2 Circumferential Stress

Similarly Figure 2.2 considers the circumferential stresses caused by internal pressure,  $P_i$ , acting on the thin cylinder of thickness  $t$ , length  $L$ , and diameter  $D$ .

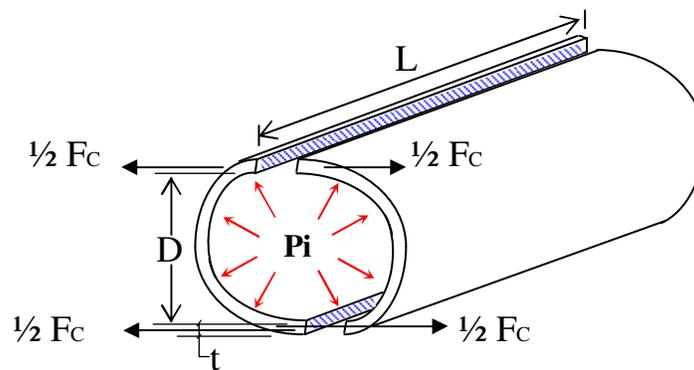


Figure 2.2 Circumferential forces acting on thin cylinder under internal pressure

Circumferential force,  $F_C = P_i \times D \times L$

Area resisting  $F_C$ ,  $a = 2. L \times t$   
(Shade area)

$$\text{Circumferential Stress, } S_C = \frac{\text{Circumferential Force, } F_C}{\text{Resisting Area, } a}$$

$$S_C = \frac{P_i \cdot D}{2 \cdot t}$$

In term of thickness,

$$t_C = \frac{P_i \cdot D}{2 \cdot S_C}$$

We call this equation as Circumferential Stress Thickness Equation.

### 3.2.3 Longitudinal Stress versus Circumferential Stress

Comparing the both thickness equations due to the longitudinal stress and circumferential stress, with a specific allowable stress, pressure and fixed diameter, the required wall thickness to withstand the internal pressure,  $P_i$ , for circumferential stress will twice that required for the longitudinal stress. Circumferential stress in the thin wall will be the governing stress and hence the Circumferential Stress Thickness Equation ( $t_C$ ) is used.

### 3.2.4 Circumferential Stress Thickness Equation and 1-Foot Method

From the Circumferential Stress Thickness Equation, replace the internal pressure,  $p_i$  to the hydrostatic pressure due to product liquid head ( $\rho gh$ ), consider the effective head at 0.3 m height ( $H - 0.3$ ), and consider the corrosion allowance ( $C.A$ ) by adding in to the equation as per Figure 2.3. The minimum required thickness from the 1-Foot method can be now be derived.

$$t = \frac{\cancel{P} \cdot D}{2 \cdot S_c} + C.A \quad \Rightarrow \quad t = \frac{4.9D(H - 0.3)G}{S_d} + C.A$$

Allowable design stress,  $S_d$

**Figure 2.3 Circumferential Stress Thickness equation to 1-Foot method equation**

### 3.2.5 Shell Design Thickness Calculation

The design calculation for the shell wall thickness is attached in Appendix B. The calculation result for the shell wall thickness is summaries in Table 2.1 and Figure 2.4.

Course No.	Width (mm)	Height (mm)	t.design (mm)	t.hydro. (mm)	t.min (mm)	tsc. (mm)
1	2440	20,700	27.30	21.60	27.30	28
2	2440	18,260	24.40	19.02	24.40	25
3	2440	15,820	21.49	16.43	21.49	22
4	2440	13,380	18.58	13.85	18.58	19
5	2440	10,940	15.67	11.26	15.67	16
6	2440	8,500	12.77	8.68	12.77	13
7	2020	6,060	9.86	6.10	11	11
8	2020	4,040	7.45	3.96	11	11
9	2020	2,020	5.04	1.82	11	11

**Table 2.1 Shell wall design thickness summary**

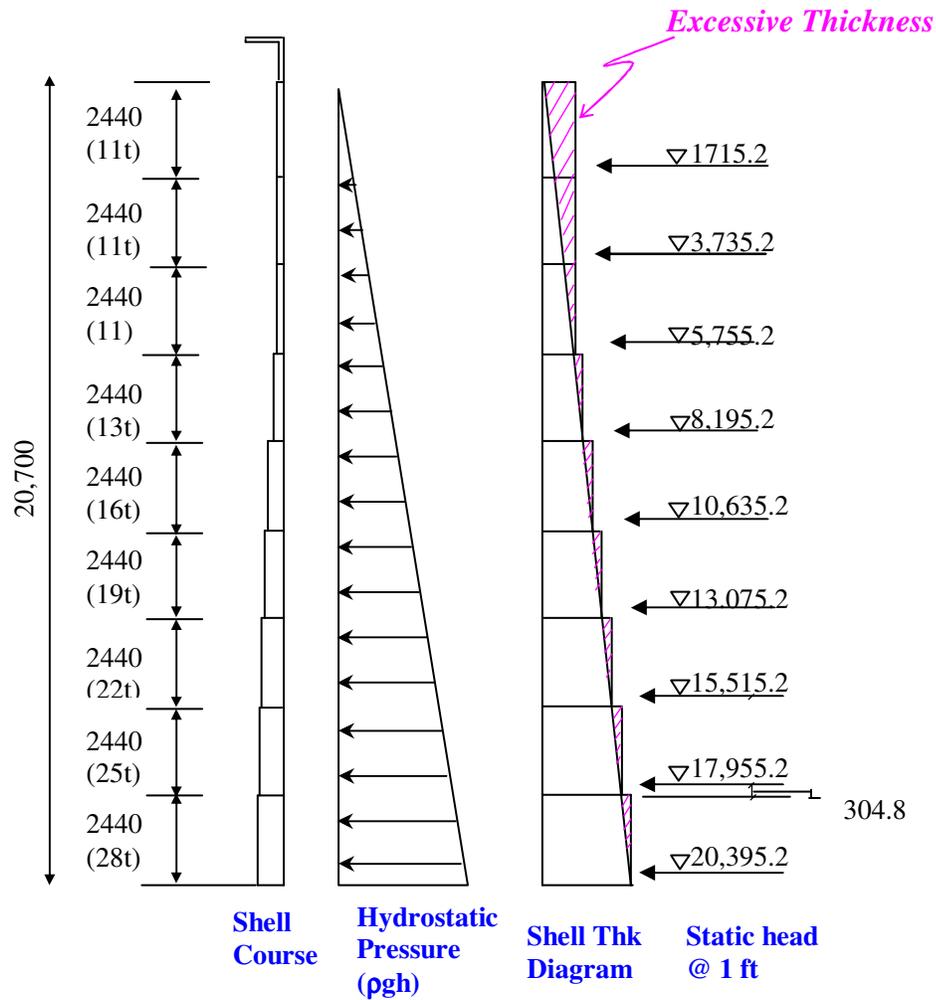
Where,

t.design = Minimum required thickness due to design condition,

t.hydro. = Minimum required thickness due to hydrostatic test,

t.min = The greater value of t,design and t.hydro., and

tsc = Actual thickness used.



**Figure 2.4 Diagrammatic sketch of shell wall with design thickness**

From the 1-Foot equation, it can be seen that the minimum required shell thickness is directly proportional to the liquid static height; hence the shell thickness diagram shall follow the same shape profile with the hydrostatic pressure due to the design liquid height as shown in Figure 2.4. However it is impractical to construct the tank with the taper thickness, therefore different shell course with different thickness is used. The use of courses with diminishing thickness will have the effect that, at the joint between two adjacent courses, the thicker lower course provides some stiffening to the top, thinner course and this causes an increase in stress in the upper part of the lower course and a reduction in stress in the lower part of the upper course. API 650 (2007) assumes that the

reduction in stress in the upper course reaches a maximum value at one foot (300 mm) above the joint and it is at this point, on each course from which the effective acting head is measured [Bob, 2004]. This shows how the 1-Foot method was employed.

### 3.2.6 Top Stiffener and Intermediate Wind Girder Design

#### 3.2.6.1 Top Stiffener/ Top Wind Girder

Stiffener rings of top wind girder are to be provided in an open-top tank to maintain the roundness when the tank is subjected to wind load. The stiffener rings shall be located at or near the top course and outside of the tank shell. The girder can also be used as an access and maintenance platform. There are five numbers of typical stiffener rings sections for the tank shell given in API 650 (2007) and they are shown in Figure 2.5 [API 650, 2007].

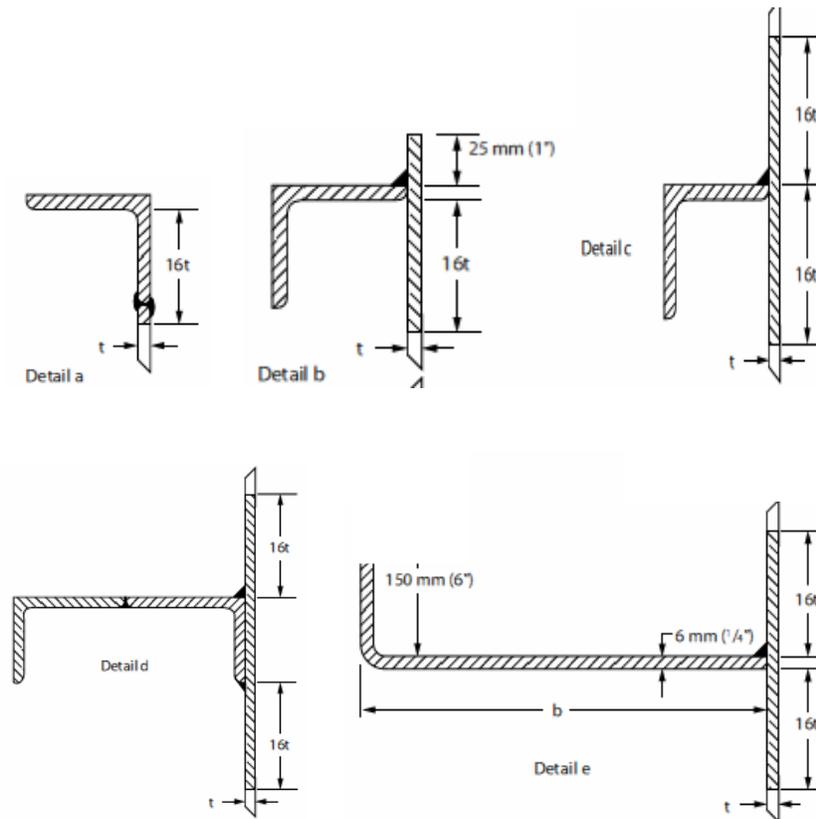


Figure 2.5 Typical stiffener ring section for ring shell

The requirement in API 650 (2007) stated that when the stiffener rings or top wind girder are located more than 0.6 m below the top of the shell, the tank shall be provided with a minimum size of 64 x 64 x 4.8 mm top curb angle for shells thickness 5 mm, and with a 76 x 76 x 6.4 mm angle for shell more than 5 mm thick. A top wind girder in my tank is designed to locate at 1 m from the top of tank and therefore for a top curb angle of size 75 x 75 x 10 mm is used in conjunction with the stiffener detail a) in Figure 2.5. The top wind girder is designed based on the equation for the minimum required section modulus of the stiffener ring [API 650, 2007].

$$Z = \frac{D^2 H_2}{17} \left( \frac{V}{190} \right)^2$$

Where

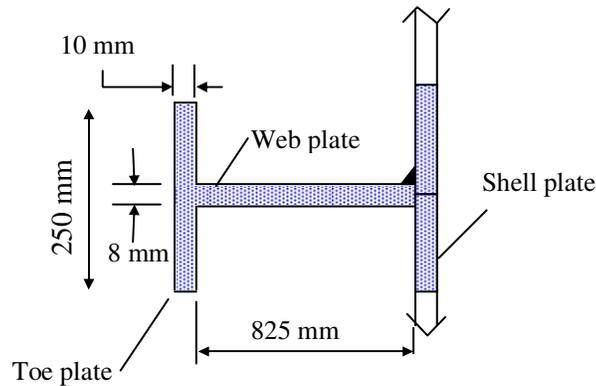
$Z =$  Minimum required section modulus,  $\text{cm}^3$

$D =$  Nominal tank diameter, m

$H_2 =$  Height of the tank shell, in m, including any freeboard provided above the maximum filling height

$V =$  design wind speed (3-sec gust), km/h

The term  $\frac{D^2 H}{17}$  on the equation is based on a wind speed of 190 km/h and therefore the term  $\left( \frac{V}{190} \right)^2$  is included in the equation for the desire design wind speed. The design calculation for the top wind girder is attached in Appendix B section 4.0. From the design calculation, a fabricated Tee-girder of size T 825 x 250 x 8 x 10 with toe plate length 250 mm, web plate length 825 mm, toe plate thickness 10 mm and web plate thickness 8mm is used. The detail of the Tee-girder used for the top wind girder is shown in Figure 2.6.



**Figure 2.6 Fabricated Tee Girder for Top Wind Girder**

With the design wind speed of 140 km/h, nominal tank diameter of 39,000 mm and height of tank shell 20,700 mm, the minimum required section modulus for the top wind girder was found to be 1,007,140 mm<sup>3</sup> and the available section modulus for Tee girder T 825 x 250 x 8 x 10 is 2,655,662 mm<sup>3</sup>. Therefore the selected girder size is sufficient.

Accordance to API 60 (2007) clause 5.9.5, support shall be provided for all stiffener rings when the dimension of the horizontal leg or web exceeds 16 times the leg or web thickness [API 650, 2007]. The supports shall be spaced at the interval required for the dead load and vertical live load. The web length of 825 mm had exceeded the 16 times of its thickness (16 x 8 = 128 mm), supports for the girders will be provided.

### **3.2.6.2 Intermediate Wind Girder**

The shell of the storage tank is susceptible to buckling under influence of wind and internal vacuum, especially when in a near empty or empty condition. It is essential to analysis the shell to ensure that it is stable under these conditions. Intermediate stiffener or wind girder will be provided if necessary.

To determine whether the intermediate wind girder is required, the maximum height of the un-stiffened shell shall be determined. The maximum height of the un-stiffener shell will be calculated as follows [API 650, 2007]:

$$H_1 = 9.47t \sqrt{\left(\frac{t}{D}\right)^3 \left(\frac{190}{V}\right)^3}$$

Where

$H_1$  = Vertical distance, in m, between the intermediate wind girder and top wind girder

$t$  = Thickness of the top shell course, mm

$D$  = Nominal tank diameter, m

$V$  = design wind speed (3-sec gust), km/h

As stated in earlier section 3.25, the shell is made of up diminishing thickness and it makes the analysis difficult. The equivalent shell method is employed to convert the multi-thickness shell into an equivalent shell having the equal thickness as to the top shell course. The actual width of each shell course is changed into a transposed width of each shell course having the top shell course thickness by the following formula [API 650, 2007]:

$$W_{tr} = W \sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5}$$

Where

$W_{tr}$  = Transposed width of each shell course, mm

$W$  = Actual width of each shell course, mm

$t_{uniform}$  = Thickness of the top shell course, mm

$t_{actual}$  = Thickness of the shell course for which the transpose width is being calculated, mm

The sum of the transposed width of the courses will be the height of the transformed shell ( $H_2$ ). The summary of transform shell height is shown in Figure 2.7.

Shell course	Shell thickness tsc.cor (mm)	Actual width W (mm)	Transposed width Wtr (mm)
1	25.00	2,440	141
2	22.00	2,440	195
3	19.00	2,440	281
4	16.00	2,440	431
5	13.00	2,440	725
6	10.00	2,440	1,397
7	8.00	2,020	2,020
8	8.00	2,020	2,020
9	8.00	2,020	2,020
10	-	-	-
11	-	-	-
12	-	-	-
13	-	-	-
14	-	-	-
15	-	-	-
Height of transformed shell, H2 =			9,230 mm

**Figure 2.7 Height of transform shell**

If the height of transformed shell is greater than the maximum height of un-stiffened shell, intermediate wind girder is required. The total number intermediate wind girder required can be determined by simply divide the height of transformed shell with the maximum un-stiffened shell height. The maximum un-stiffened shell height is calculated to be 9,182 mm which is less than the transformed shell height; hence an intermediate wind girder is required. The detail calculation of the intermediate wind girder is attached in Appendix B section 5.0.

Similarly, minimum required section modulus of the intermediate wind girder has to be determined. The same equation in the top wind girder can be used, but instead of the total shell height  $H_2$ , the vertical distance between the intermediate wind girder and top wind girder is used. The equation will become [API 650, 2007]:

$$Z = \frac{D^2 H_1}{17} \left( \frac{V}{190} \right)^2$$

Where

$Z =$  Minimum required section modulus,  $\text{cm}^3$

$D =$  Nominal tank diameter, m

$H_2 =$  Height of the tank shell, in m, including any freeboard provided above the maximum filling height

$V =$  design wind speed (3-sec gust), km/h

The minimum required section modulus for the intermediate wind girder was calculated to be  $225,812 \text{ mm}^3$  and a fabricated Tee-girder of size T 405 x 150 x 8 x 8 with toe plate length 150 mm, web plate length 405 mm, toe plate thickness 8 mm and web plate thickness 8 mm is used. The available section modulus for intermediate Tee girder is  $863,143 \text{ mm}^3$  and proven that the selected girder size is sufficient. The detail of the selected intermediate Tee-girder is shown in Figure 2.8.

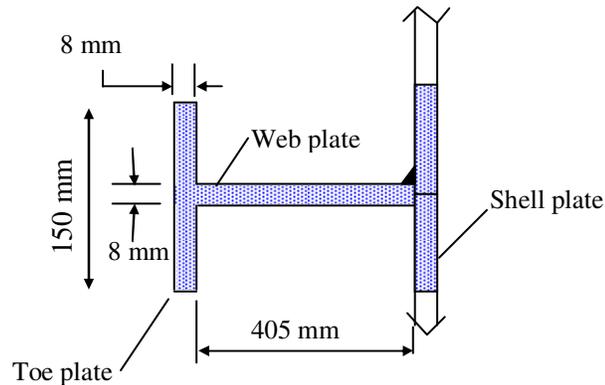
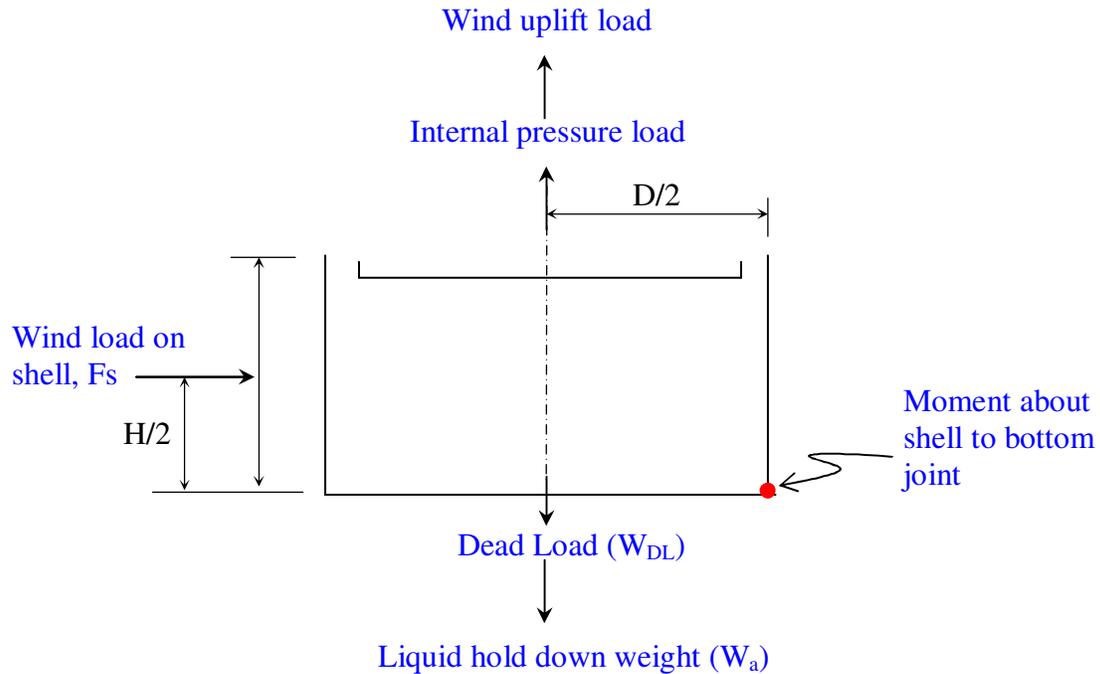


Figure 2.8 Fabricated Tee Girder for Intermediate Wind Girder

### 3.2.7 Overturning Stability against Wind Load

The overturning stability of the tank shall be analyzed against the wind pressure, and to determine the stability of the tank with and without anchorage. The wind pressure used in the analysis is given as per API 650 (2007). The design wind pressure on the vertical projected areas of cylindrical surface area ( $w_s$ ) shall be  $0.86 \text{ kPa} (V/190)^2$  and  $1.44 \text{ kPa} (V/190)^2$  uplift on horizontal projected area of conical surface ( $w_r$ ). These design wind

pressure are in accordance with American Society of Civil Engineer - ASCE 7 for wind exposure Category C [ASCE 7, 2005]. The loading diagram due to the wind pressure on the floating roof tank is shown in Figure 2.9.



**Figure 2.9** Overturning check on tank due to wind load

The wind load ( $F_s$ ) on the shell is calculated by multiplying the wind pressure  $w_s$  to the projected area of the shell, and the wind load ( $F_r$ ) on the roof will be zero as the roof will be floating on the liquid into the tank, where there will be no projected area for the roof.

As per API 650 (2007), the tank will be structurally stable without anchorage when the below uplift criteria are met [API 650, 2007].

- i.  $0.6 M_w + M_{pi} < M_{DL} / 1.5$
- ii.  $M_w + 0.4 M_{pi} < (M_{DL} + M_F) / 5$

Where

$M_{pi}$  = moment about the shell-to-bottom from design internal pressure ( $P_i$ ) and it can be calculated by the formula  $\left(\frac{1}{4} \pi \times D^2 \times P_i\right) \times \frac{1}{2} D$ .

$M_w$  = Overturning moment about the shell-to-bottom joint from horizontal plus vertical wind pressure and is equal to  $F_r.L_r + F_s.L_s$ .  $F_r$  and  $F_s$  is the wind load acting on the roof and shell respectively and  $L_r$  and  $L_s$  is the height from tank bottom to the roof center and shell center respectively.

$M_{DL}$  = Moment about the shell-to-bottom joint from the weight of the shell and roof supported by the shell and is calculated as  $0.5 D \cdot W_{DL}$ . The weight of the roof is zero since the roof is floating on the liquid.

$M_F$  = Moment about the shell-to-bottom joint from liquid weight and is equal to  $\left(\frac{wa \times \pi \times D}{1000}\right) \times \frac{D}{2}$ .

The liquid weight ( $wa$ ) is the weight of a band of liquid at the shell using a specific gravity of 0.7 and a height of one-half the design liquid height  $H$ .  $Wa$  will be the lesser of  $0.90 H.D$  or  $59 \times t_b \sqrt{F_{by} \times H}$ .  $F_{by}$  is the minimum specified yield stress of the bottom plate under the shell and  $t_b$  is the thickness of Bottom plate under the shell.

The detail calculation for the overturning stability against wind load is in Appendix B section 6.0. The calculation had shown that both the uplift criteria are met and the tank will be structurally stable even without anchorage. A summarized result is shown in Figure 2.10.

$$\begin{aligned} 0.6 M_w + M_{pi} &= \mathbf{4,345,020,578} < MDL / 1.5 \\ M_w + 0.4 M_{pi} &= \mathbf{7,241,700,964} < (M_{DL} + M_F) / 2 \end{aligned}$$

**Figure 2.10 Summary Result for Overturning Stability against wind load**

### 3.2.8 Seismic Design

The seismic design of the storage tank is accordance to API 650 (2007) – Appendix E. There are three major analyses to be performed in the seismic design, and they are:

- i) Overturning Stability check - The overturning moment will be calculated and check for the anchorage requirement. The number of anchor bolt required and the anchor bolt size will also be determined based on the overturning moment.
- ii) Maximum base shear
- iii) Freeboard required for the sloshing wave height – It is essential for a floating roof tank to have sufficient freeboard to ensure the roof seal remain within the height the tank shell.

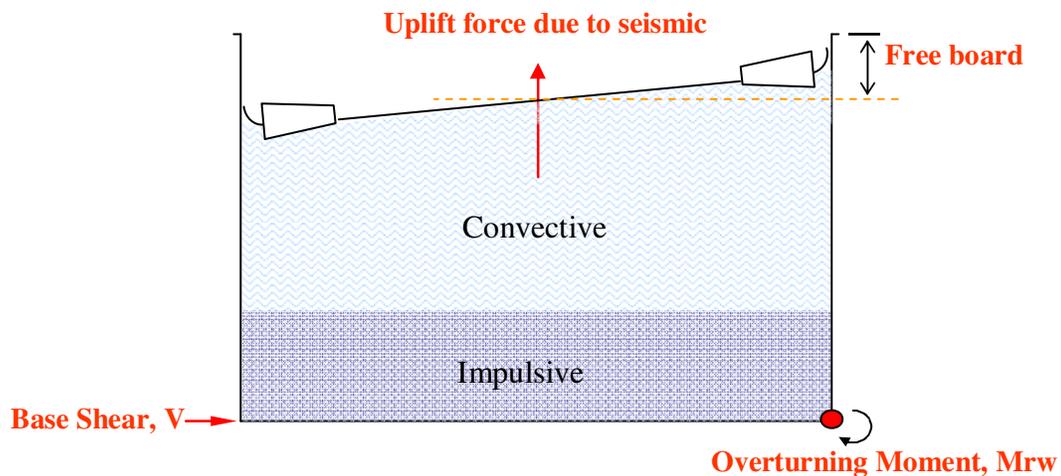


Figure 2.11 Seismic Diagram for a Floating Roof Tank

The behavior of liquid in a vertical cylindrical container when subjected to an earthquake was clarified by G.W. Houser in his paper “Earthquake Pressures on Fluid Containers” and the theory is now widely used and also applied in API 650 (2007). The seismic design addressed in API 650 (2007) Appendix E is based on the Allowable Stress Design (ASD) Method with the specific load combination and the ground motion requirements are derived from ASCE 7, which is based on a maximum considered earthquake ground motion defined as the motion due to an event with a 2% probability of exceed within a 50-year period [API 650, 2007]. The pseudo-dynamic design procedures are based on the response spectra analysis methods and two response modes of the tank and its content – impulsive and convective are considered.

The impulsive component is the part of the liquid in the lower part of the tank which moves with the tank as though it were a solid. It experiences the same accelerations and displacement as the tank. The convective component is the part of the liquid in the upper part of the tank which is free to form waves or to slosh. It has a much longer natural frequency time than the impulsive portion. The detail of the convective frequency is discussed in section 3.2.8.4. The impulsive mode is based on a 5% damped response spectral and 0.5% damped spectral for the convective mode. Impulsive and convective shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method.

The tank is presumed to be rigid but this is not exactly true. This presumption is normally made for the ambient tanks and it provides answers of sufficient accuracy, but only to the tank shell. This seismic design is only apply to the tank shell, seismic design of floating roofs is beyond the API 650 (2007) scope and it will be a challenge for engineer to analyses the seismic effect on the floating roof.

### 3.2.8.1 Site Geometry Design Data for Seismic Design

The site geometry design data for seismic design to be used in the analysis are as follow:

- i) Seismic Peak Ground Acceleration,  $S_p = 0.3g$
- ii) Importance Factor,  $I = 1.50$
- iii) Site Class = D
- iv) Seismic Group,  $SUG = III$

This tank is to be built and installed in Turkmenistan, which is outside the U.S.A region and not defined in ASCE 7. For site not defined in ASCE 7, API 650 (2007) defined the following substitution [API 650, 2007]:

- For 5% damped spectral response acceleration parameter at short period of 0.2 sec,  $S_s = 2.5 S_p$
- For 5% damped spectral response acceleration parameter at period of 1.0 sec,  $S_1 = 1.25 S_p$

### 3.2.8.2 Overturning Stability

The seismic overturning moment at the base of the tank shall be the SRSS summation of the impulsive and convective components multiply by the respective moment arms to the center of action of the forces.

For tanks supported by the concrete ring wall, the equation for calculating the ringwall moment,  $M_{rw}$  is as follow [API 650, 2007]:

$$Mrw = \sqrt{[Ai(WiXi + WsXs + WrXr)]^2 + [Ac(WcXc)]^2}$$

Where

$Ai$  = Impulsive design response spectrum acceleration coefficient, %g

$Ac$  = Convective design response spectrum acceleration coefficient, %g

$Wi$  = Effective impulsive portion of liquid weight, N

$Ws$  = Total weight of the tank shell and appurtenances, N

$Wr$  = Total weight of fixed tank roof including framing, knuckles, any permanent attachments and 10% of the roof design snow load, N

$Wc$  = Effective convective (sloshing) portion of liquid weight, N

$Xi$  = Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for ring wall moment, m

$Xs$  = Height from the bottom of the tank shell to the shell's center of gravity, m

$Xr$  = Height from the bottom of the tank shell to the roof and roof appurtenances center of gravity, m

$Xc$  = Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the convective liquid force for ring wall moment, m

This overturning moment is important for the mechanical to design the anchorage requirement and determine the minimum the number and size of the anchor bolt for the storage tank. It is also important to the civil engineer to design the tank foundation in which the tank is being supported.

### 3.2.8.3 Design Spectral Accelerations

The spectral acceleration parameters are given in the equation below and they are based on the response spectrum pictured in Figure 2.12. The parameter in equation are defined the section 8.2.8.4.

- Impulsive spectral acceleration parameter,  $A_i$  [API650, 2007]:

$$A_i = S_{DS} \left( \frac{I}{R_{wi}} \right) = 2.5Q.Fa.S_o \frac{I}{R_{wi}}$$

But,  $A_i \geq 0.007$

And, site class E and F only,:

$$A_i = 0.5S_1 \left( \frac{I}{R_{wi}} \right) = 0.875S_p \frac{I}{R_{wi}}$$

- Convective spectral acceleration parameter,  $A_c$  [API650, 2007]:

For  $T_c \leq T_L$ ,

$$A_c = K S_{D1} \left( \frac{1}{T_c} \right) \left( \frac{I}{R_{wc}} \right) = 2.5K.Q.Fa.S_o \left( \frac{T_s}{T_c} \right) \left( \frac{I}{R_{wc}} \right)$$

For  $T_c > T_L$ ,

$$A_c = K S_{D1} \left( \frac{T_L}{T_c^2} \right) \left( \frac{I}{R_{wc}} \right) = 2.5K.Q.Fa.S_o \left( \frac{T_s T_L}{T_c^2} \right) \left( \frac{I}{R_{wc}} \right)$$

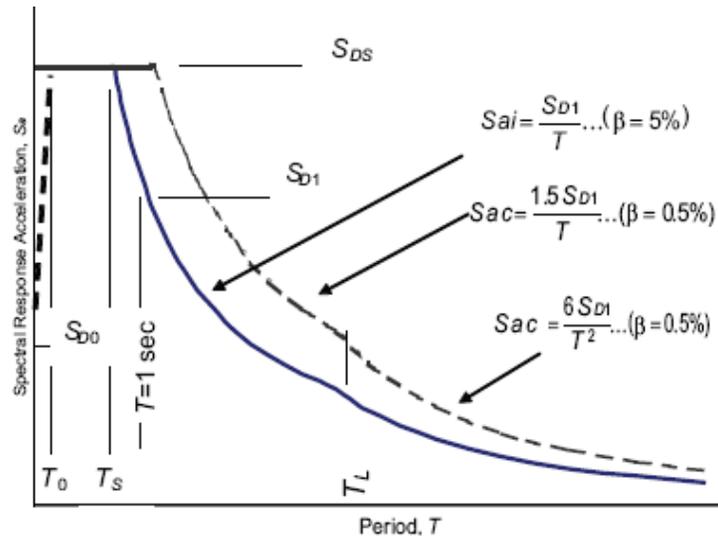


Figure 2.12 Design Response Spectral for Ground-Supported Liquid Storage Tanks [API650, 2007]

### 3.2.8.4 Parameter Required for Seismic Design

- i) Convective (Sloshing) Period,  $T_c$

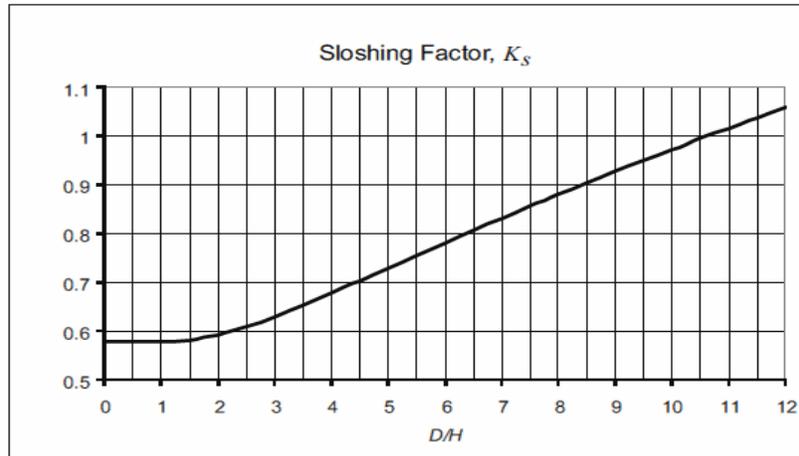
The first mode sloshing wave period ( $T_c$ ), in second is calculated by the following equation [API650, 2007].

$$T_c = 1.8 K_s \cdot \sqrt{D}$$

Where  $K_s$  = sloshing period coefficient and is defined as

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}}$$

Or it can also be determined from the figure 2.13.



**Figure 2.13 Sloshing Period Coefficient,  $K_s$  [API650, 2007]**

ii) Regional-dependent transition period for longer period ground motion,  $T_L$

It was defined in API 650 (2007) that for regions outside U.S.A,  $T_L$  shall be taken as 4 seconds [API650, 2007].

iii) Scaling Factor,  $Q$

The scaling factor,  $Q$  was defined to be taken as 1.0 in API 650 (2007) unless it was otherwise defines in the regulatory requirement where ASCE 7 does not apply [API650, 2007].

iv) Acceleration-based site coefficient (at 0.2 sec period),  $F_a$

The acceleration- based site coefficient at 0.2 second period,  $F_a$  was determined directly from the Table 2.2.

Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods					
Site Class	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

<sup>a</sup>Site-specific geotechnical investigation and dynamic site response analysis is required.

**Table 2.2 Value of  $F_a$  as a Function of Site Class [API650, 2007]**

For site class of D and  $S_s$  as 2.5  $S_p$ , where  $S_p = 0.3g$ ,  $S_s = 0.75$ , therefore  $F_a$  is taken as 1.2.

v) Velocity-based site coefficient (at 1.0 sec period),  $F_v$

Similarly, the velocity-based site coefficient at 1.0 second period,  $F_v$  was determined directly from the Table 2.3.

Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1 Sec Periods					
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

<sup>a</sup>Site-specific geotechnical investigation and dynamic site response analysis is required.

**Table 2.3 Value of  $F_v$  as a Function of Site Class [API650, 2007]**

For site class of D and  $S_1$  as 1.25  $S_p$ , where  $S_p = 0.3g$ ,  $S_1 = 0.375$ ,  $F_a$  is to be interpolate between the value in  $S_1 = 0.3$  and  $S_1 = 0.4$ . The interpolated value for  $F_v$  is 1.65.

vi) Response modification factors for ASD Methods,  $R_w$

The ASD response modification factors,  $R_{wi}$  for impulsive and  $R_{wc}$  for convective are normally defined by the regulations, and if these value are not defined by the regulations, the values defined in API 650 (2007) shall be used. There is no value defined by the regulation for this project, therefore value from API 650 (2007) will be used. The response modification factors for ASD method defined in API 650 (2007) as shown in Table 2.4.

Anchorage System	$R_{wi}$ (Impulsive)	$R_{wc}$ (Convective)
Self-anchored	3.5	2
Mechanically - anchored	4	2

**Table 2.4 Response Modification Factors for ASD Methods [API650, 2007]**

The tank was designed to be mechanically anchored, therefore the response modification factors for Impulsive ( $R_{wi}$ ) is 4 and for Convective ( $R_{wc}$ ) is 2.

The design parameters are summarized in the Table 2.5 and the spectral accelerations can be calculated.

	Impulsive	Convective
Q	1	
Fa	1.2	
Fv		1.65
I	1.5	
$R_w$	4	2
$T_c$	6.63 s	
$T_L$	4 s	
$S_o$	0.3	
$S_{DS}$	0.9	
$S_{D1}$	0.6187	

**Table 2.5 Summary of design parameter**

Impulsive Spectral Acceleration,

$$A_i = S_{DS} \left( \frac{I}{R_{wi}} \right) = 2.5 Q.Fa.S_o \frac{I}{R_{wi}} = 0.34\% g$$

Convective Spectral Acceleration,

( $T_c > T_L$ )

$$A_c = K S_{D1} \left( \frac{T_L}{T_c^2} \right) \left( \frac{I}{R_{wc}} \right) = 2.5 K.Q.Fa.S_o \left( \frac{T_s.T_L}{T_c^2} \right) \left( \frac{I}{R_{wc}} \right) = 0.0633\% g$$

And the response spectrum curve is plotted as shown in Figure 2.14.

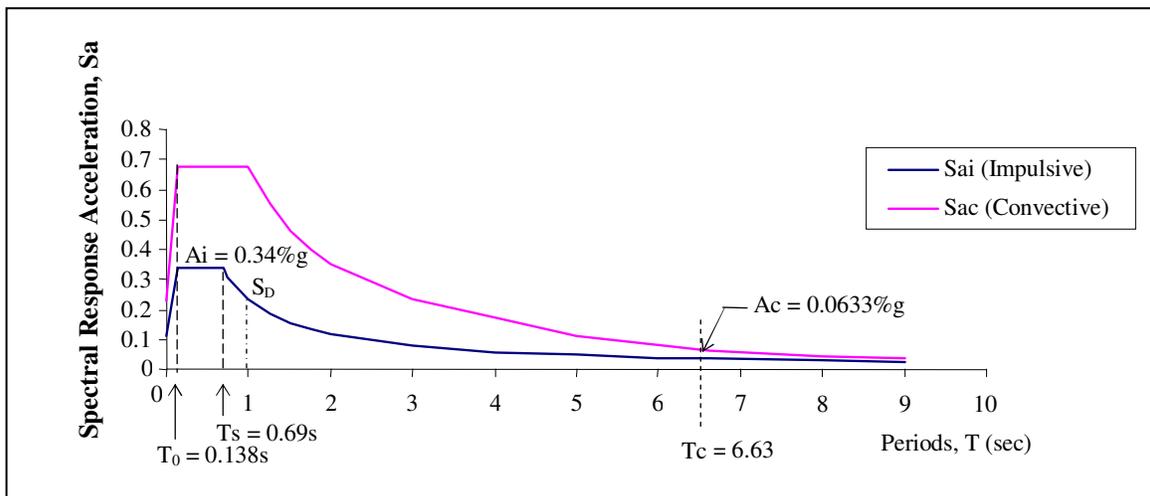


Figure 2.14 Response Spectrum Curve

### 3.2.8.5 Effective Weight of Product

The effective weights  $W_i$  and  $W_c$  are determined by multiplying the total product weight,  $W_p$  by the weight ratio ( $W_i / W_p$ ) and ( $W_c / W_p$ ) respectively as per equation below. These equations are originally developed by Housner and it is now employed by the API 650 (2007). The relationships between the equations are also graphically illustrated in Figure 2.15. The proportion of the product liquid in the impulsive and convective

portions is a function of the tank shape and the calculation methods will be different for short tanks with  $D/H$  greater than 1.333 and for tall tanks with  $D/H$  less than 1.333.

- For effective impulsive weight,

When  $D/H \geq 1.333$ ,

$$W_i = \frac{\tanh\left(0.866 \frac{D}{H}\right)}{0.866} \cdot W_p$$

When  $D/H < 1.333$ ,

$$W_i = \left[1.0 - 0.218 \frac{D}{H}\right] \cdot W_p$$

- For effective convective weight,

$$W_c = 0.230 \frac{D}{H} \tanh\left(\frac{0.367H}{D}\right) \cdot W_p$$

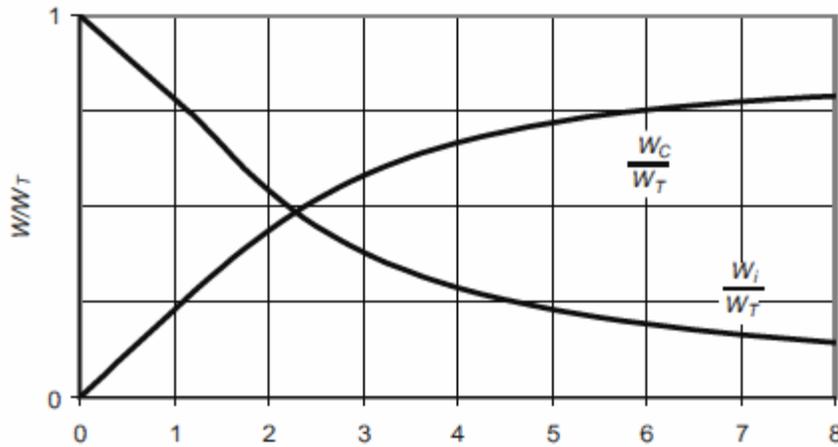


Figure 2.15 Effective weight of Liquid ratio [API650, 2007]

### 3.2.8.6 Center of Action for Effective Lateral Forces

The moment arm from the base of the tank to the center of action for the equivalent lateral forces from the liquid has to be defined for the overturning moment. The center of action for the impulsive lateral forces for the tank shell, roof and appurtenances is assumed to act through the center of gravity of the component.

The heights from the bottom of the tank shell to the center of action of the lateral force seismic force applied to the effective weights  $W_i$  and  $W_c$ ,  $X_i$  and  $X_c$  are determined by multiplying the maximum design liquid height  $H$  by the ratio  $(X_i / H)$  and  $(X_c / H)$  respectively as per equation below [API 650, 2007]. The relationships between the equations are also graphically illustrated in Figure 2.16.

- For impulsive force,

When  $D/H \geq 1.333$ ,

$$X_i = 0.375 H$$

When  $D/H < 1.333$ ,

$$X_i = \left[ 0.5 - 0.094 \frac{D}{H} \right] \cdot H$$

- For convective force,

$$X_c = \left[ 1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D} \sinh\left(\frac{3.67H}{D}\right)} \right] \cdot H$$

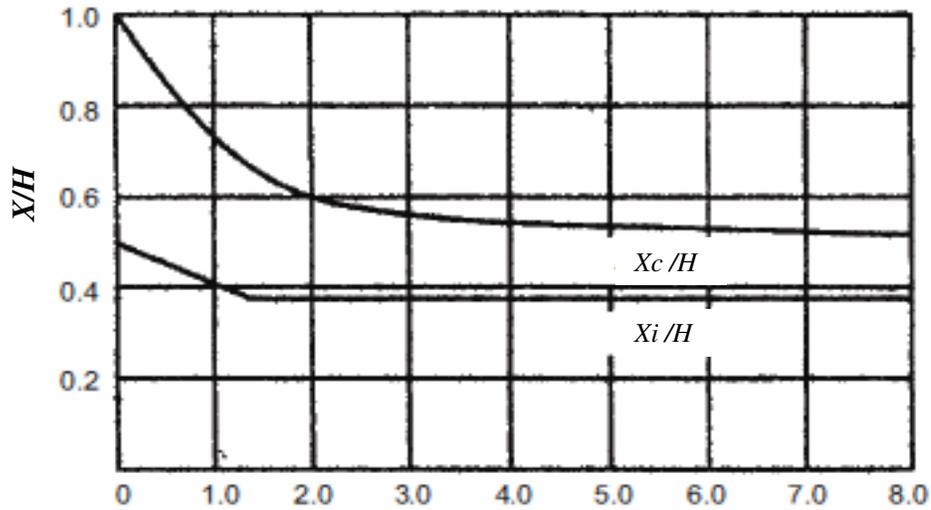


Figure 2.16 Center of Action for Effective Forces [API650, 2007]

### 3.2.8.7 Ring Wall Moment

The ring wall moment,  $M_{rw}$  now can be determine after all the parameters in 3.2.8.3 to 3.2.8.6 are defined, this moment is the portion of the total overturning moment that acts at the base of the tank shell perimeter and is used to determined loads on a ring wall foundation, the tank anchorage forces, and to check the longitudinal shell compression.

### 3.2.8.8 Base Shear Force

The seismic base shear is defined as the SRSS combination of the impulsive and convective components with the following equation [API 650, 2007].

$$V = \sqrt{V_i^2 + V_c^2}$$

Where

$V_i$  = Impulsive force and is defined as

$$V_i = A_i ( W_s + W_r + W_f + W_i ),$$

$V_c$  = Convective force and is defined as

$$V_c = A_c \cdot W_c$$

And

$W_i$  = Effective impulsive portion of liquid weight, N

$W_s$  = Total weight of the tank shell and appurtenances, N

$W_r$  = Total weight of fixed tank roof including framing, knuckles, any permanent attachments and 10% of the roof design snow load, N

$W_f$  = Total weight of the tank bottom, N

$W_c$  = Effective convective (sloshing) portion of liquid weight, N

Not that the tank is a floating roof tank, therefore  $W_r = 0$  and the total weight of the tank roof is added to the weight of the tank content, as the roof is floating on the liquid.

The base shear force and the ring wall moment due to the seismic effect is summarized the seismic moment and force diagram in Figure 2.17.

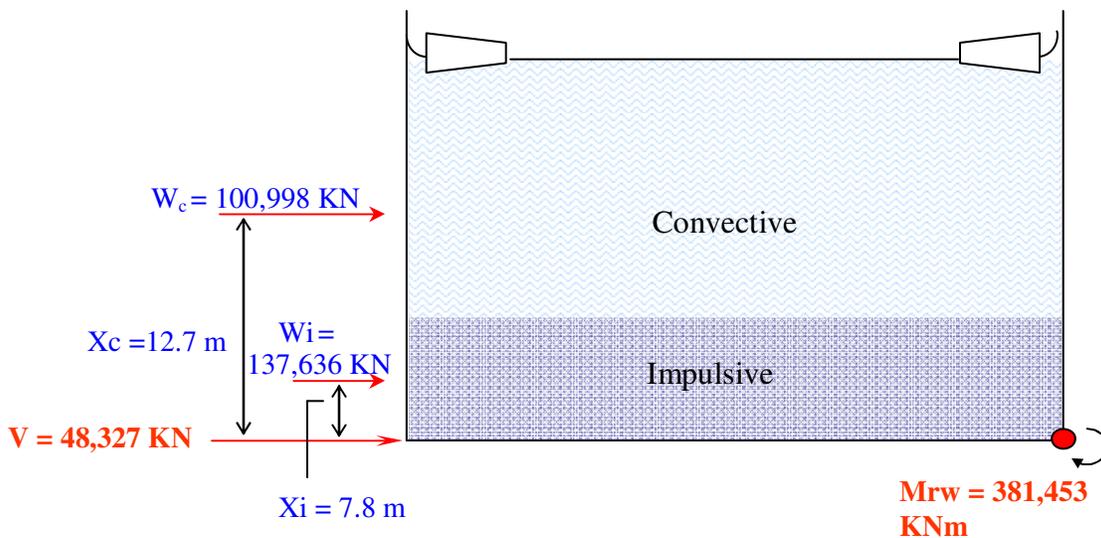


Figure 2.17 Seismic Moment and Force Diagram

### 3.2.8.9 Resistance to Overturning

There are three resisting components to resist against the overturning due to the seismic; they are the i) anchorage, ii) annular plate width which sits directly under the first shell course and iii) the shell compression at the bottom of the shell.

#### i) Anchorage requirement

The resistance to the design ring wall overturning moment at the base of the shell will be provided by the weight of the tank shell, weight of the roof reaction,  $W_{rs}$ , by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or provided by the mechanical anchorage devices.

The anchorage requirement is checked by the Anchorage Ratio,  $J$ , and the anchorage ratio criteria in Table 2.6 will determine whether the tank can be self-anchored or mechanically anchored.

Anchorage Ratio, $J$	Criteria
$J \leq 0.785$	No calculated uplifted under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \leq 0.154$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular plate if $L < 0.035D$ is not controlling or add mechanical anchorage.

Table 2.6 Anchorage Ratio Criteria [API650, 2007]

The anchorage ratio, J is determined as follow [API650, 2007]:

$$J = \frac{Mrw}{D^2 [wt(1-0.4A_v) + wa - 0.4w_{int}]}$$

Where

$wt$  = Weight of tank shell & portion of roof supported by shell and is define as

$$wt = \frac{W_s}{\pi \cdot D} + w_{rs}$$

$wa$  = Resisting force of annulus which is defined as

$$7.9ta \sqrt{F_y \cdot H \cdot G_e} \leq 1.28 H \cdot D \cdot G_e$$

And  $F_y$  = Min. specified yield strength of bottom annulus, = 241 N/mm<sup>2</sup>

$H$  = Maximum design product level, m

$G_e$  = Effective specific gravity including vertical seismic effect  
 =  $G \cdot (1 - 0.4 A_v)$  ;  $G = 1$ , Specific gravity

$A_v$  = Vertical earthquake acceleration coefficient

= 0.7 (as defined in Site Design Data)

$W_{int}$  = Uplift due to product pressure

= 0 (for floating roof tank )

$w_{rs}$  = Roof load acting on shell, including 10% of specified snow load

= 0 (for floating roof)

The anchorage ratio was found to be 2.19 which is more the 1.54; therefore the tank has to be mechanically anchored. Anchor bolt will have to be design and sized up.

ii) Annular plate requirement

Before going into the anchor bolt design, annular plate width shall be checked for stability due to seismic. For the thickness of the bottom plate or annular plate ( $t_a$ ) under the shell is thicker than the remainder, the minimum projection of the supplied thicker annular plate inside the tank wall shall be at least equal to  $L$  and not more than 0.035 times the tank nominal diameter, and

$$L = 0.01723 \cdot t_a \sqrt{\frac{F_y}{H \cdot G_e}} \quad (450 \leq L \leq 0.035D)$$

The minimum annular width,  $L$  was calculated as 1,108.57 mm and the actual width used is 1,200 mm. Hence the annular plate width is sufficient for the seismic loading.

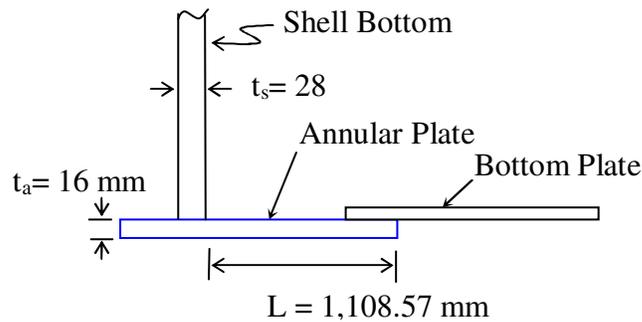


Figure 2.18 Annular Plate Requirement

iii) Shell Compression

The maximum shell longitudinal compression stress at the bottom of the shell for the mechanical-anchored tanks is determined by the below formula, and  $t_s$  is the thickness of the bottom shell course less corrosion allowance [API 650, 2007].

$$\sigma_c = \left( w_t (1 + 0.4A_v) + \frac{1.273Mrw}{D^2} \right) \frac{1}{1000t_s}$$

The calculated maximum longitudinal shell compression stress has to be less than the allowable stress  $F_c$ , which can be determined as follow [API 650, 2007]:

$$\text{When } \frac{GHD^2}{t^2} \geq 44, \quad F_c = \frac{83ts}{D}$$

$$\text{When } \frac{GHD^2}{t^2} < 44, \quad F_c = \frac{83ts}{2.5D} + 7.5\sqrt{(G \cdot H)}$$

$$\text{And} \quad F_c < 0.5F_{ty}$$

The maximum longitudinal shell compression stress,  $\sigma_c$  is calculated to be 12.69 N/mm<sup>2</sup>,  $\frac{GHD^2}{t^2}$  is 40.22 which is less than 44; and  $F_c$  is found to be 57.94 N/mm<sup>2</sup> which is less than 0.5 time the minimum specific yield stress of the bottom shell,  $F_{ty}$ . Therefore, the tank is structurally stable.

### 3.2.8.10 Anchorage Design

As the tank was found to be structurally unstable and cannot be self-anchored for the design load, the tank has to be anchored with the anchor bolts. The anchor bolts are sized to provide the minimum anchorage resistance, the design uplift load on the anchor bolts due to the seismic is determined by the following [API650, 2007]:

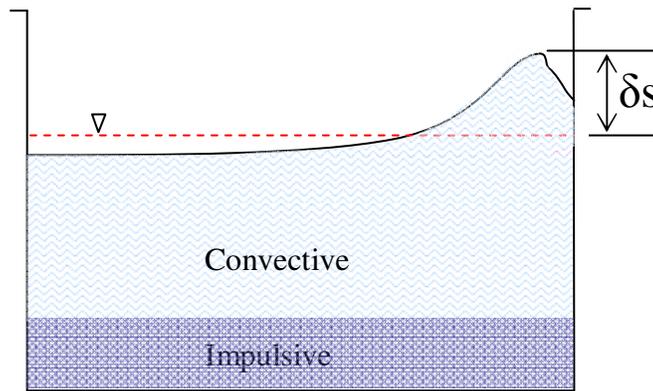
$$w_{AB} = \left( \frac{1.273Mr_w}{D^2} - w_i(1 - 0.4Av) \right) + w_{int}$$

And it calculated to be 36.796 KN. The tensile stresses in the anchor bolt which the uplift load applied on have to be check against the allowable tensile strength, which is 0.8 time its specify yield stress,  $S_y$ . The material used for the anchor bolts is the high strength bolt SA 320 Gr.L7, with the minimum specific yield stress of 551.5 N/mm<sup>2</sup>, and the allowable

tensile strength for the bolt will be  $0.8S_y = 441.2 \text{ N/mm}^2$ . Total 86 numbers of M64 bolts are pre-selected for the design, and hence the tensile stress on each of the anchor bolt can be determined by  $\sigma_b = \frac{W_{AB}}{N \cdot A_b}$  and found to be  $161.94 \text{ N/mm}^2$ , hence proving that the selected number (N) and the anchor bolt size (A<sub>b</sub>) is sufficient.

### 3.2.8.11 Freeboard

The minimum freeboard required above the top capacity is determined by considering the sloshing of the liquid inside the tank.



**Figure 2.19 Sloshing Wave of Liquid Inside Tank**

The sloshing wave height above the product design height can be estimated by the following equation [API 650,2007]:

$$\delta s = 0.5 D \cdot A_f$$

Where, for  $T_c > T_L$  in the seismic group SUG III,

$$A_f = K \cdot S_{D1} \frac{T_L}{T_C^2} = 2.5 K \cdot Q \cdot FaSo \left( \frac{T_s T_L}{T_C^2} \right)$$

$A_f$  was found to be 0.08 and  $\delta_s$  will be 1,647 mm. Accordance to API 650 (2007), the minimum required freeboard for the SUG III tanks and shall be equal to the sloshing wave height,  $\delta_s$  [API 650, 2007].

#### **3.2.8.12 Seismic Design Summary**

The complete seismic design calculation can be found in Appendix B - section 7 at the end of the report.

### 3.3 Roof Design

There is limited procedure and rules provided for the floating roof design as most of the components; particularly the fitting and accessories in the floating roof are proprietary design. The roof design consists of roof type selection, buoyancy design, roof stress design and the fitting and accessories design and operation.

#### 3.3.1 Roof Type Selection

Different types of floating roof had been discussed in the previous chapter – literature review. Therefore it is not worth to repeat here. The pontoon type - single deck floating roof was normally used for tank diameter less than 65 m due to flexibility of the deck plate, double deck will be used for larger diameter tank as double is more rigid and stable. In view of out tank diameter of 39 m, and the cost effectiveness, the single deck floating roof was selected. Further consideration of the insulation effect of the double deck roof was also considered. As our tanks are to be built in a country with extreme winter and snow, the consideration of melting the snow from the product is essential, where the insulation effect due to the air gap between the decks plate in the double deck floating roof is not favorable.

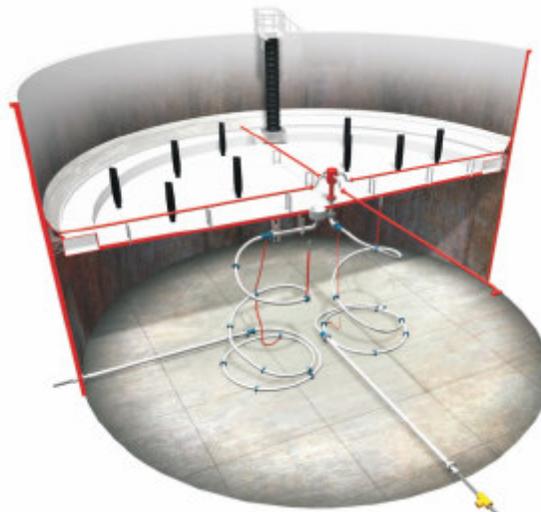
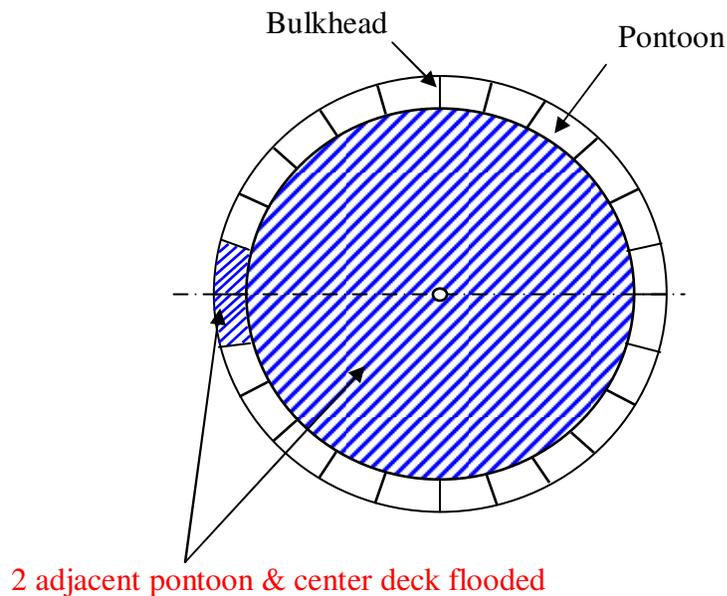


Figure 3.1 Single deck Floating roof

### 3.3.2 Pontoon and Center Deck Design

The basic requirement as stated in API 650 (2007) for the pontoon design is that the pontoon has to be designed to have sufficient buoyancy to remain on the product with the design specific gravity of 0.7 or lower for the product and inoperative of roof drain for:

- Deck plate & any two adjacent pontoon compartments punctured and flooded the center deck as per figure 3.2.
- Rainfall of 10” (250 mm) in 24 hour period over roof area.



**Figure 3.2 Center deck and 2 adjacent compartments puncture**

API 650 (2007) required all the deck plate to have a minimum nominal thickness of 5 mm and the deck of the single deck pontoon floating roof has to be designed to be in contact with the liquid during normal operation. The design shall be able to accommodate the deflection of the deck caused by trapped vapour. A nominal thickness of 8 mm was used in my center deck design, and this thickness will be verified the design calculation. Figure 3.3 shows the minimum requirement for the single deck pontoon floating roof

with the inoperative roof drain, compartments puncture and deck plate flooded and holding of 250 mm of rainfall.

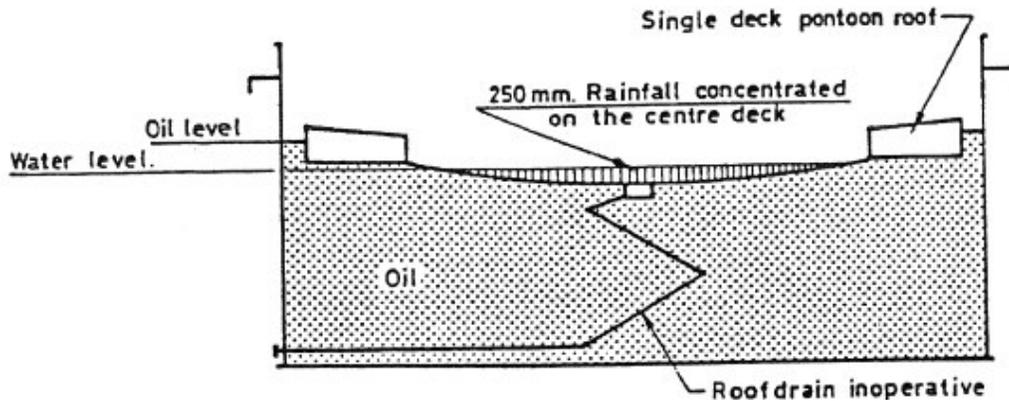


Figure 3.3 Minimum Requirement for Single Deck Pontoon Floating Roof [EEMUA 2003, vol.1, p118]

### 3.3.2.1 Roof Stress Design

Roof stress design is performed on the center deck by studying the stresses and analyzing the effects of the stresses on the roof. There are two load cases used,

- i) Dead Load Only – No flooding in center deck

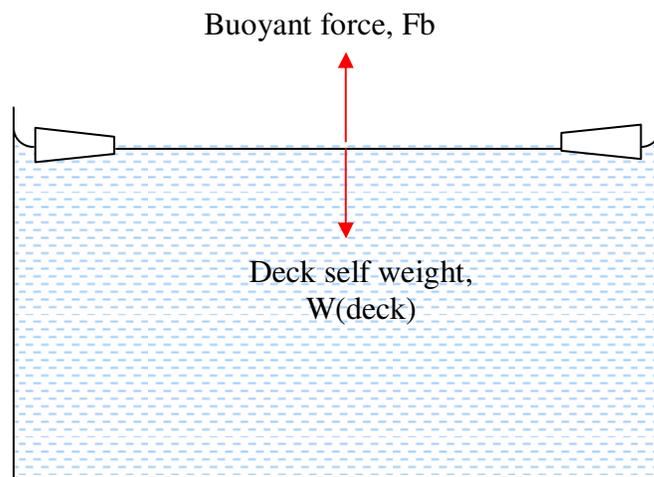


Figure 3.4 Case 1 – Dead Load Only

$$\text{Unit Lateral Pressure} = \frac{W(\text{deck}) - F_b}{\text{Deck Area}}$$

ii) Dead load plus 250 mm of rain accumulation

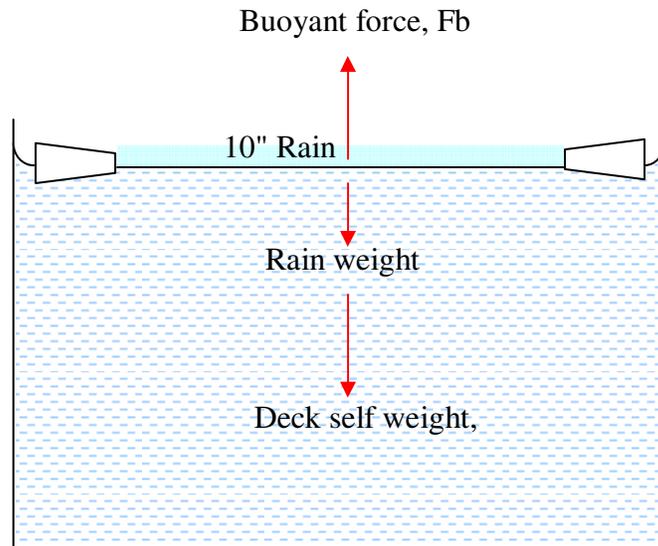


Figure 3.5 Case 2 – Dead Load + 10” Rain Accumulation

$$\text{Unit Lateral Pressure} = \frac{W(\text{deck}) + W(\text{rain}) - F_b}{\text{Deck Area}}$$

### 3.3.2.2 Effect of Large Deflection on Center Deck

When a flat plate deflects under the normal condition, the middle surface, halfway between top and bottom surfaces will remain unstressed; at other points there will be biaxial stress in the plane of the plate. When the deflection becomes larger and exceeds one-half the plate thickness, the middle surface will become appreciably strained and the stresses in it would cause defect or failure and hence it should not be ignored. This will be the case in the thin deck plate of 8 mm. Figures 3.6 (a) and (b) show the deflection of the center deck under the two cases.

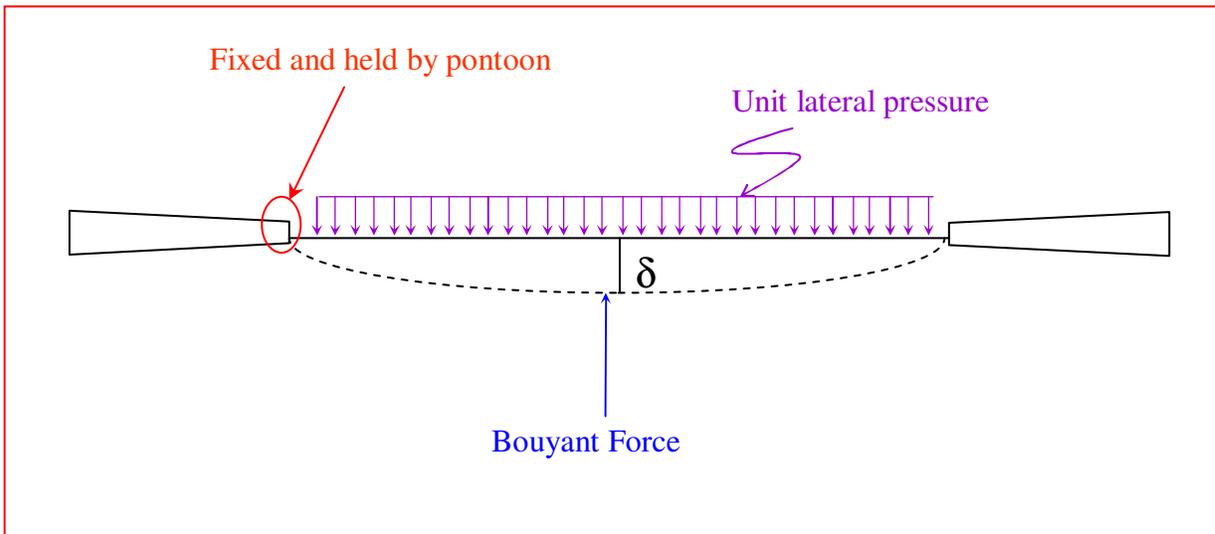


Figure 3.6 (a) Deck Deflection in Case 1

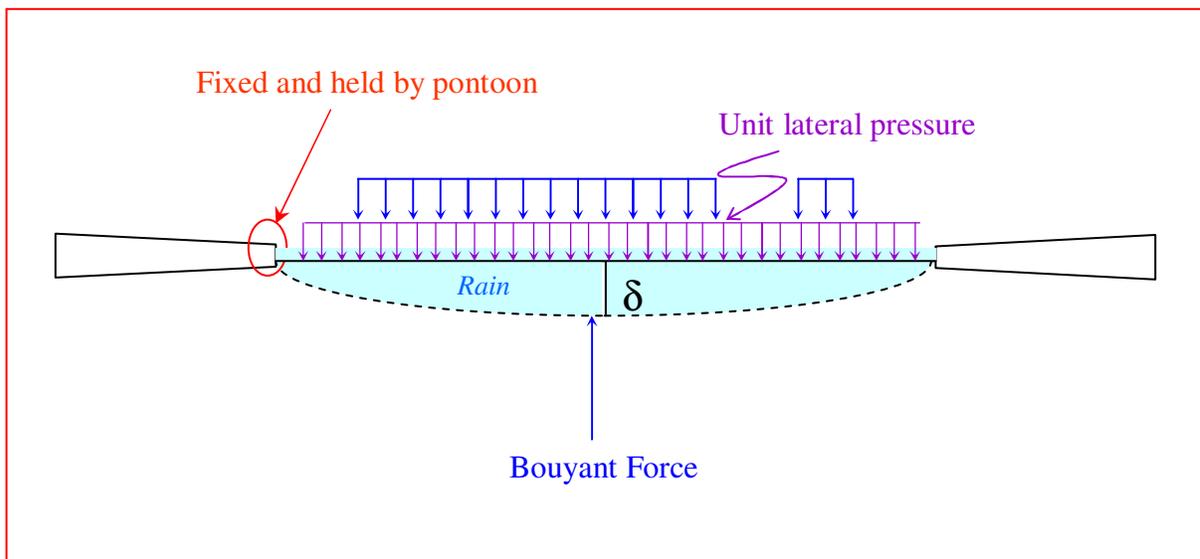


Figure 3.6 (b) Deck Deflection in Case 2

This middle surface stress is called the diaphragm stress, or direct stress, and it enables the plate to carry part of the load as a diaphragm in direct tension. This tension may be balanced by radial tension at the edges if the edges are held or by circumferential compression if the edges are not horizontally restrained. This circumferential compression may cause buckling in the thin plate.

In the large deflection of the thin plate, the plate is stiffer than indicated by the ordinary theory and the load-deflection and load-stress relation become non-linear. For circular plates, where the maximum deflection exceeded half the thickness, the below formula shall be used for more accurate and precise result [Roark, 2002].

$$\frac{q\alpha^4}{Et^4} = K_1 \frac{y}{t} + K_2 \left( \frac{y}{t} \right)^3$$

$$\frac{\sigma\alpha^2}{Et^2} = K_3 \frac{y}{t} + K_4 \left( \frac{y}{t} \right)^2$$

Where

$t =$  Thickness of plate (deck plate), mm

$\alpha =$  Outer radius of deck plate, mm

$q =$  Unit lateral pressure on deck, N/mm<sup>2</sup>

$y =$  Maximum deflection, mm

$\sigma =$  Maximum stress due to flexure and diaphragm tension combined  
 $= \sigma_b + \sigma_d$

$\sigma_b =$  Bending stress, N/mm<sup>2</sup>

$\sigma_d =$  Diaphragm stress, N/mm<sup>2</sup>

The K constants are determined in the Roark's Formula for Stress and Strain for different cases and edge condition. The center deck plate is fixed and held at its outer edge by the pontoon, hence the condition is considered as case no. 3 – edge condition fixed and held with uniform pressure q over entire plate. The constants will then be determined as below,  $\nu$  is the poisson ratio which is equal to 0.3 [Roark, 2002].

$$K_1 = \frac{5.33}{1-\nu^2} = 5.86$$

$$K_2 = \frac{2.6}{1-\nu^2} = 2.86$$

At Center :

$$K_3 = \frac{2}{1-\nu} = 2.86$$

$$K_4 = 2.86$$

At Edge :

$$K_3 = \frac{4}{1-\nu^2} = 4.40$$

$$K_4 = 1.73$$

The maximum deflection and the stresses for the both cases are summarized the Table 3.1.

	LOAD CASE 1		LOAD CASE 2	
	Deck Center	Deck Edge	Deck Center	Deck Edge
Max. Deflection, y (mm)	215.81		214.38	
$\sigma$ total (N/mm <sup>2</sup> )	35.92	62.84	33.94	59.37
$\sigma$ bending (N/mm <sup>2</sup> )	3.52	5.41	3.34	5.14
$\sigma$ diaphragm (N/mm <sup>2</sup> )	32.40	57.43	30.0	54.38

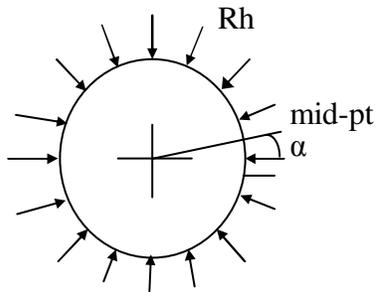
**Table 3.1 Summary Result for Maximum Deflection and Stresses in Center Deck**

### 3.3.2.3 Pontoon Stability – Pontoon Ring Design

The diaphragm stresses at the deck edge caused the tension at the outer edge of the deck; hence there will be radial force acting at the inner rim of the pontoon. The relationship between the radial force and the diaphragm stress as shown below.

$$R_h = \sigma \text{ diaphragm} \times \text{deck thickness}$$

Rh acting on the Inner Rim is modeled as load point at each mm of circumference, with a very small angle between load points approximated to uniform distributed load in the circular ring design.



Number of point loads at each mm is,

$$Nlp = \pi \times \text{Rim Diameter } (\varnothing_{ir})$$

$$\text{, and angle } \alpha = \frac{1}{2} \cdot \frac{360}{Nlp}$$

**Figure 3.7 Radial Forces Acting on Pontoon Inner Rim**

The pontoon stability due to the radial loads is designed with reference to the Roark's Formula for Stress and Strain, it is model as closed circular ring and regarded as a statically indeterminate beam and analyzed by the use of Castigliano's second theorem [Roark, 2002]. Formulas used are taken directly from the Table 9.2 in Roark's Formula for Stress and Strain, and they are based on several assumptions as listed below [Roark, 2002].

- i) The ring is of uniform cross section and has symmetry about the plane of curvature.
- ii) All loading are applied at the radial position of the centroid of the cross section. This is not the case for our pontoon ring as the radial load acting on the inner rim are in the lower position, however this assumption is of little concern for thin ring.
- iii) It is nowhere stressed beyond the elastic limit.
- iv) It is not so severely deformed as to lose its essentially circular shape.
- v) Its deflection is due primarily to bending.

Case 7 - Ring under any number of equal radial forces equally space from Table 9.2 in Roark's Formula for Stress and Strain is selected and the formulas for the bending moment and circumferential tensile force between and at the load point are as follow [Roark, 2002]:

i) At mid-point:

$$\text{Bending Moment, } M_m = \frac{Rh \cdot Do}{4} \left( \frac{1}{\sin \alpha} - \frac{1}{\alpha} \right)$$

$$\text{Cir. Tensile Force, } T_m = \frac{Rh}{2 \sin \alpha}$$

ii) At load-point:

$$\text{Bending Moment, } M_r = \frac{Rh \cdot Do}{4} \left( \frac{1}{\alpha} - \frac{1}{\tan \alpha} \right)$$

$$\text{Cir. Tensile Force, } T_r = \frac{Rh}{2 \tan \alpha}$$

The pontoon ring stability is checked against the pontoon properties. Figure 3.8 shows the basic geometry for the pontoon and the results are summarized in the Table 3.2. The pontoon section modulus,  $Z_a$  is calculated to 27,019,626 mm<sup>3</sup>.

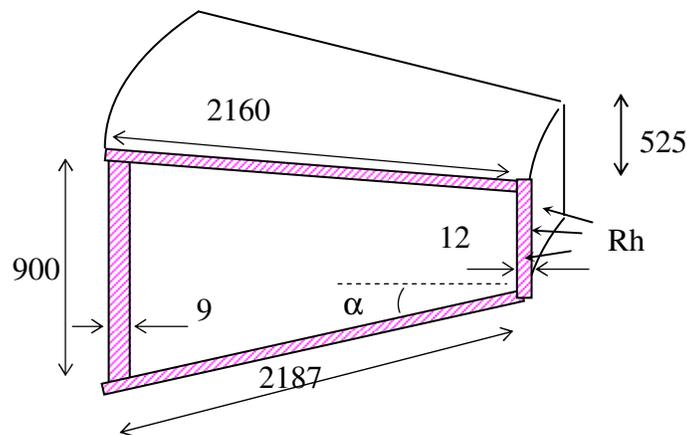


Figure 3.8 Sectional Detail of Pontoon

<b>RING STABILITY CHECK</b>	<b>LOAD CASE 1</b>		<b>LOAD CASE 2</b>	
	MID - POINT	LOAD- POINT	MID - POINT	LOAD- POINT
Bending Moment (Nmm)	19.14	-38.29	18.08	-36.15
Circ. Force (N)	7,867,429	7,867,429	7,429,209	7,429,209
Bending Stress (N/mm <sup>2</sup> )	0.0000007	-0.000001	0.0000007	-0.000001
Circ. Stress (N/mm <sup>2</sup> )	159.98	159.98	151.07	151.07
Allow. Bending Stress (N/mm <sup>2</sup> )	183	183	183	183
Allow. Axial Stress (N/mm <sup>2</sup> )	165	165	165	165
Unity Check	0.97	0.97	0.92	0.92
Condition	<b>OK</b>	<b>OK</b>	<b>OK</b>	<b>OK</b>

**Table 3.2 Summary Result for Pontoon Ring Stability**

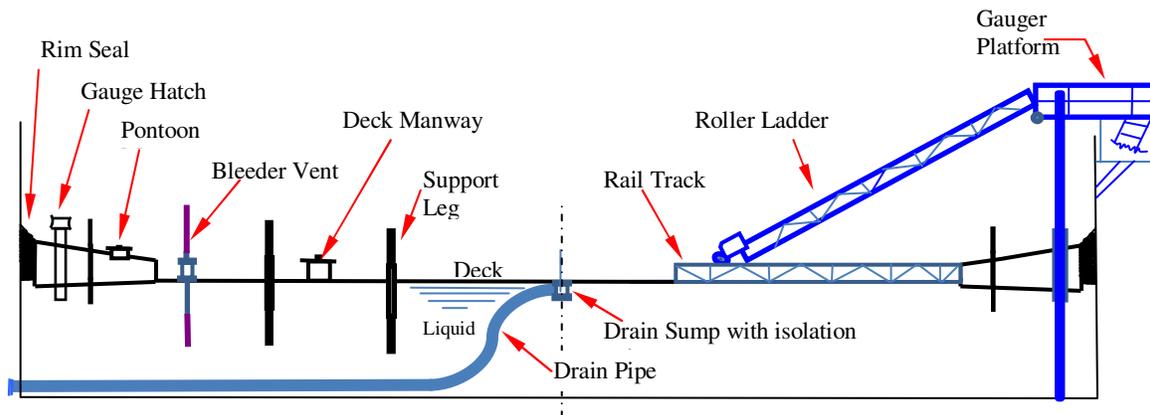
Where,

$$\text{Unity} = \frac{\text{Actual Bending Stress}}{\text{Allow. Bending Stress}} + \frac{\text{Actual Circ. Stress}}{\text{Allow. Comp. Stress}}$$

The complete design calculation on the roof stress design is attached in Appendix B section 5 of this thesis report.

### 3.3.3 Fitting and Accessories Design

Figure 3.9 shows the typical standard accessories and fitting for single deck floating roof which are essential for the operation of the floating roof tank. Each of the fitting and accessories has its own unique importance; malfunction of any one of the fitting would cause roof failure and potentially leading to fatality. The minimum requirement for the roof fitting had been outlined in the Table 1.8 discussed in the Literature Review chapter.



**Figure 3.9 Standard Fitting and Accessories for Single Deck Roof**

### 3.3.3.1 Roof Seal System

As discussed in the chapter 2.4.2 principles of the floating roof, there will be a 200 mm of gap between the inside of tank shell and the outer rim of the floating roof pontoon. The main purpose of the roof seals are to close up the gap between pontoon & shell wall, hence preventing the escape of vapor from the tank product to the atmosphere and minimize the amount of rain and pollutant entering the product. The seals are also to allow irregularities of the tank and roof construction and to account any radial or lateral movement of the roof due to the wind and seismic. Therefore the seal must be flexible enough to take in all these purposes.

Normally there will be two types of seals installed in the floating roof tank; they are i) primary seal and ii) secondary seal. There are several different types of primary seal available in the market today, and the appropriate seal has to be selected for suit the tank service. The seals design are the proprietary design by the seal company, the most that the engineer or tank designer can do is to study on each of the different seal and based on the previous experience to do the seal selection.

i) Primary Seal

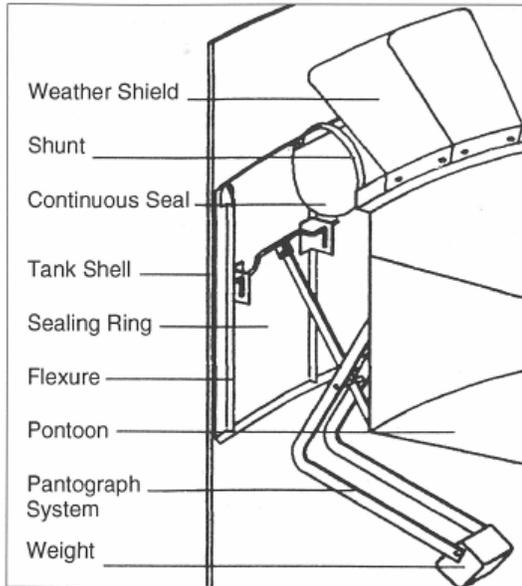
The functions of the primary seal are to minimize vapour loss, centralize the floating roof and exclude snow, rain from the rim gap. Primary seal could be in metallic (Mechanical Shoe Seal) or non metallic (Resilient Filled Seal) type.

- Mechanical (Metallic) Shoe Seal

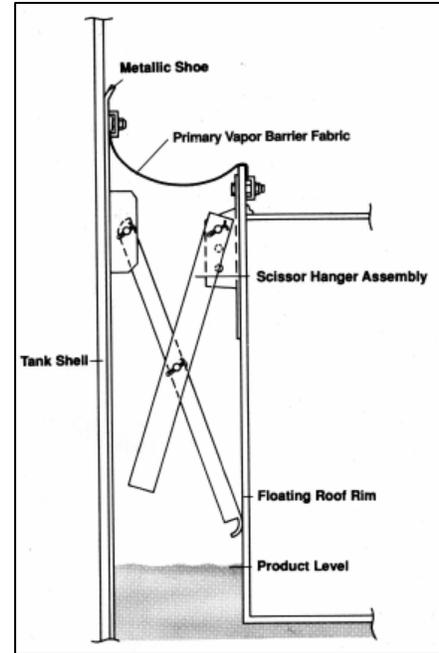
The Mechanical Shoe Seal which was recommended in API 650 (2007) has two different kinds of seals, which are Pantograph Hanger and Scissor Hanger.

Pantograph Hanger as shown in Figure 3.10 is the conventional mechanical seals, it consists of a galvanized steel or stainless steel sealing ring with the bottom located below the liquid surface, a vapour tight fire-resistant continuous seal to close the rim space, and stainless steel shunts for lightning protection. The sealing ring was supported by the weighted pantograph system which the steel weights activate the tank lever system, pressing the sealing ring against the tank shell, ensuring the sealing ring is held in constant contact with the tank shell. The shoe plate is designed with Flexures built into the sheet at intervals of approximately 550mm to ensure conformity with the tank shell and allow expansion and contraction. This seal is able to provide a rim space variation of  $\pm 130$  mm in a nominal 200 mm rim space.

Scissor Hanger as shown in Figure 3.11 was introduced to the market in the recent years. Different seal supplier could have different name for it. The design principles are basically similar to the Pantograph Hanger; it is the pusher bar to push the shoe plate instead of the counter weight. Scissor Hanger is more much simple design and economic compared to the Pantograph Hanger, also the easier installing and assembly without any hot welding work.



**Figure 3.10 Pantograph Hanger**  
*(Courtesy of VACONSEAL)*

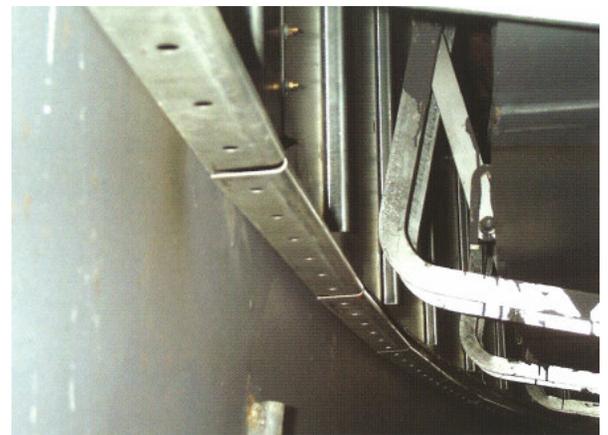


**Figure 3.11 Scissor Hanger**  
*(Courtesy of HMT)*

Figure 3.12 and Figure 3.13 shows the complete assembled and the end section of the Pantograph Hanger respectively.



**Figure 3.12 Completed Assembled Pantograph Hanger**  
*(Courtesy of WB)*

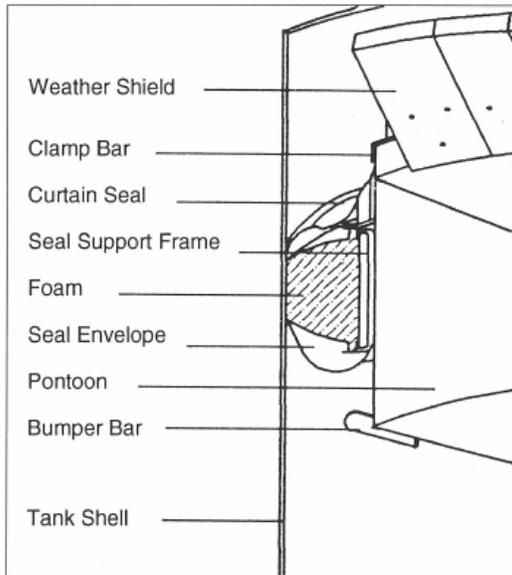


**Figure 3.13 End Section Pantograph Hanger**  
*(Courtesy of WB)*

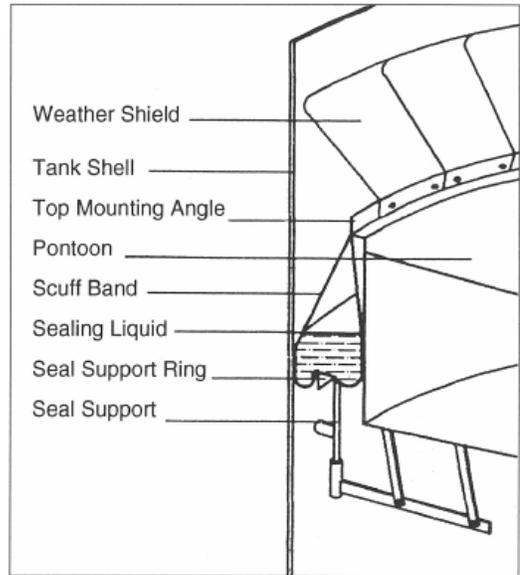
- Resilient Filled (Non-Metallic) Seal

The resilient filled seal can be of the foam filled or liquid filled. Figure 3.14 shows the foam filled and Figure 3.15 shows the liquid filled.

In the foam filled seal, the mechanical force is obtained by taking a compressible foam material and inserting it between the floating roof rim and the tank shell. Resistance to the scuffing action of the roughened tank shell plates is achieved by wrapping the resilient foam in an envelope of reinforced plastic sheet or rubber sheet. The foam and envelope may be mounted in a number of variants, where the lower part of the seal touches the stored liquid, the seal is said to be liquid mounted, and if it is mounted above the liquid, it is vapour mounted. The liquid mounted seal has better vapour conservation characteristics.



**Figure 3.14 Foam-Filled Seal**  
(Courtesy of VACONOSEAL)



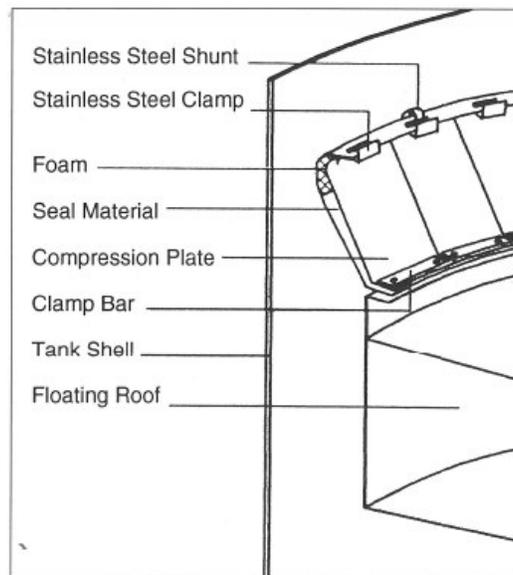
**Figure 3.15 Liquid-Filled Seal**  
(Courtesy of VACONOSEAL)

In the liquid filled seal, a looped envelope of reinforced rubber sheet is supported in the rim gap and the envelope is filled with a neutral liquid such as kerosene. By virtue of its depth and density the liquid spreads the envelope and exerts a force against the tank shell. The envelope is normally ribbed and a tube may be fitted to contain the kerosene.

After the study of the above seal system, the Mechanical Shoe Seal Scissor type was selected for its highly reputed performance, lower cost and simple installation. It was recommended by the API 650 (2007) and the liquid filled resilient seal was prohibited by some of the oil company.

ii) Secondary Seal

Secondary seal is mounted on top of the primary seal, it reduces vapour loss which in turn cost saving, enhanced safety by protection against rim fires, environmental protection with less odour and compliance with the air standards and it significantly reduces the amount of rainwater entering the tank contents by running down the shell. Figure 3.16 show one kind of the secondary seal.



**Figure 3.16 Secondary Seal (Courtesy of VACONOSEAL)**

### 3.3.3.2 Roof Seal Material

It is essential to select the correct material for the primary and secondary rim seals. The basic requirement of the seal material is the chemical resistance, which is related to the stored product, the ultraviolet resistance in which the seal expose to direct sunlight and the material has to be flame retardant.

The primary seals should always be hydro-carbon resistance since they are in direct contact with the product and product vapour and the top coat of the secondary seals shall be ultraviolet resistant and flame retardant. The tip structure of the secondary seals which slides along the tank shell would preferably be made of two kinds of material, which is hydrocarbon resistance material at the bottom section and Ultraviolet resistance at the top section. Some common materials for the selected product are listed in the Table 3.3 and the properties of the common material are shown in Table 3.4.

Fluid Stored	Seal Material
Crude Oil	Fluoropolymers, urethane, nitrile
Refined Products	Fluoropolymers, urethane, urethane laminate, fluoroelastomers, or Buna-N-Vinyl
Gasoline/ MTBE blend	Fluoropolymers, nitrile

**Table 3.3 Common Material for Select Product**

Material	Resistance Against		Flame Retardant?
	Hydrocarbons	UV light	
Vition ® (FPM)/ nylon (PA)	Very Good	Very Good	Yes
Teflon ® (PTFE)/ glass	Very Good	Very Good	Yes
Neoprene (CR)/ calcium silicate	Reasonable	Good	No
Polyurethane (EU)/ nylon (PA) or polyester (TPE-E)	Good	Good	No
PVC-nitrile (PVC-NBR)/ nylon (PA) or polyester (TPE-E) or glass	Good	Reasonable	No
Nitrile (NBR)/ Nylon (PA) or polyester (TPE-E)	Reasonable	Poor	No

**Table 3.4 Properties of Common Seal Material [EEMUA 2003, vol.1, p118]**

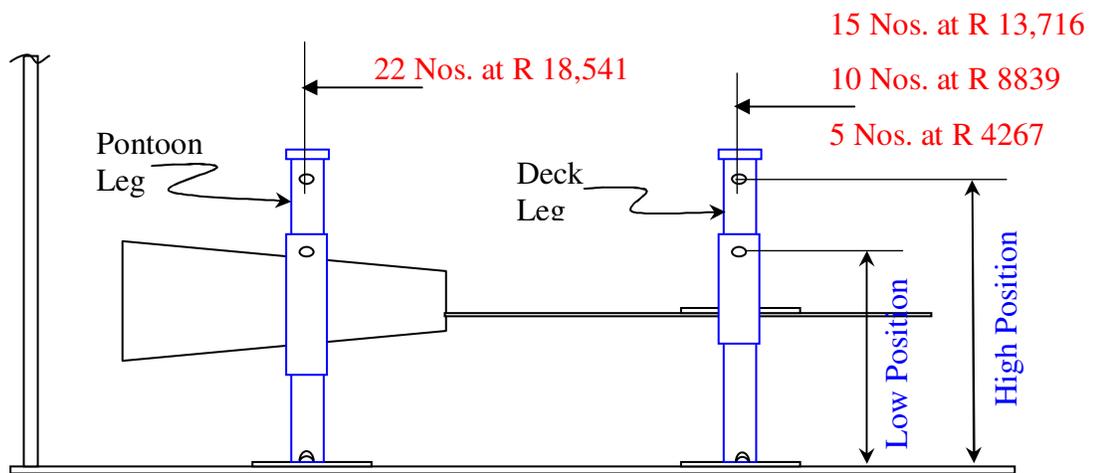
### 3.3.3.3 Roof Support Leg

Roof support legs are provided in the floating roof tank to support the roof when landed and keep the roof away from any tank appurtenances that locate at or near bottom of the tank such as inlet and outlet connection, mixers, heating coil and drainage system. The supports legs are adjustment in height to provide both a low operating position and a high cleaning position.

The basic requirement for the roof support legs had been discussed in the Literature Review in chapter 2.18.2. In designing the roof support legs, the number of support legs required for a single deck roof can be roughly approximated before a structural check on the legs is performed. There will be two type of roof support which is the pontoon support leg and the deck support leg. For the pontoon support leg, one leg per 6 m of tank circumference was approximated, and for the centre deck support leg, for tanks diameter up to 60 m, one leg per 34 m<sup>2</sup> of center deck area and for tanks diameter larger than 60 m one leg per 26 m<sup>2</sup> of center deck area was approximated.

The supports legs are to be designed to carry only the weight of the roof and a uniform live load of 1.2 KN as specified in API 650 (2007) [API 650, 2007], but not the weight of any accumulated rain water on the deck. Therefore it is important to ensure that drain out all the rain accumulation before landing the roof.

Numbers and location of the support legs for the floating roof was as shown in Figure 3.17. Standard pipe are used to design and fabricate the support legs and the pips size used are 3” Schedule 80 which has a thickness of 7.62 mm.



**Figure 3.17 Number and Location of Support Legs**

The compressive stress in each support leg at each radius location was determined and checked against the allowable stress as per AISC standard [ANSI/AISC 360, 2005] using the slenderness ratio. The complete stress design calculation for the roof support leg is attached in Appendix B Section 6. The summary stress result was tabulated in Table 3.5 and it shows that the actual stresses of all the legs are less than the allowable stress hence proven that the pre-selected number and size of the support legs are sufficient.

Leg at radius	No. of leg	Actual stress, (N/mm <sup>2</sup> )	Allowable stress, (N/mm <sup>2</sup> )	RESULT
4267.00	5.00	25.18	75.08	OK
8839.00	10.00	24.70	75.08	OK
13716.00	15.00	21.59	75.08	OK
18541.00	22.00	31.33	74.62	OK

**Table 3.5 Summary Result for Roof Support Legs**

### 3.3.3.4 Venting System

The venting system is designed to API 2000 (1998) – Venting Atmospheric and Low-Pressure Storage Tanks [API 2000, 1998]. It should not be over design; venting requirement shall be at minimal to prevent vapour loss. Automatic Bleeder Vent is the only venting fitting installed on the floating roof. They only vent the air to and from under of a floating roof during filling and emptying. The bleeder vent is simply a short piece of steel pipe fabricated with a push rod inside attached to the top cover or stopper.

#### 3.3.3.4.1 Operation of Bleeder Vent

Automatic bleeder vents/ valves only come into operation when the floating roof is landed and tank is drained down or tank is filled up. It allows product movement, where during in-breathing, it allows air to enter space under the roof as product drain out from tank, hence avoid vacuum. Similarly during out-breathing, it allows the air under the roof to escape when tank is filled up, hence avoid vapour pocket and pressure formation.

Operation of the automatic bleeder vent can be explained by the Figures 3.18 (a) and (b) for emptying (In-Breathing) and Figures 3.19 (a) and (b) for filling in (Out-Breathing).

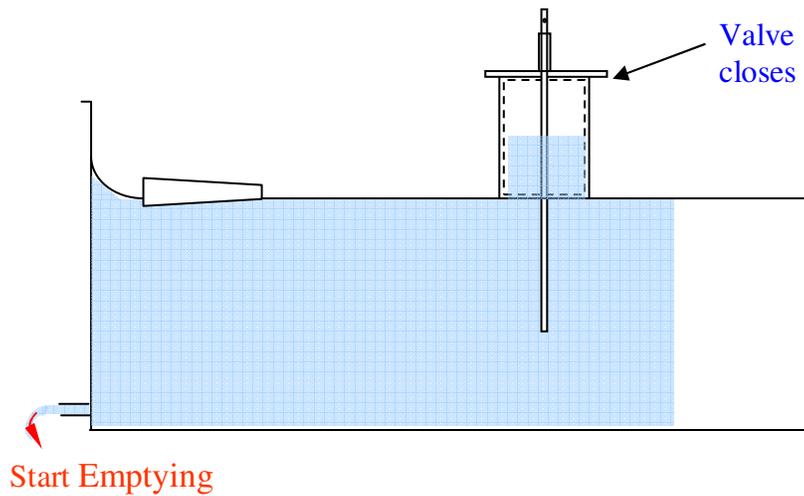


Figure 3.18 (a) Operating of Bleeder Vent during In-Breathing (Starting)

In the case of emptying (In-breathing), the roof is floating on the product when the tank start emptying and the valve is intially closed. The product continue flowing out of the tank till the push rod in the valve touches the tank floor before the support legs, pushing the valve opens and letting air flowing in freely, venting the space beneath the deck.

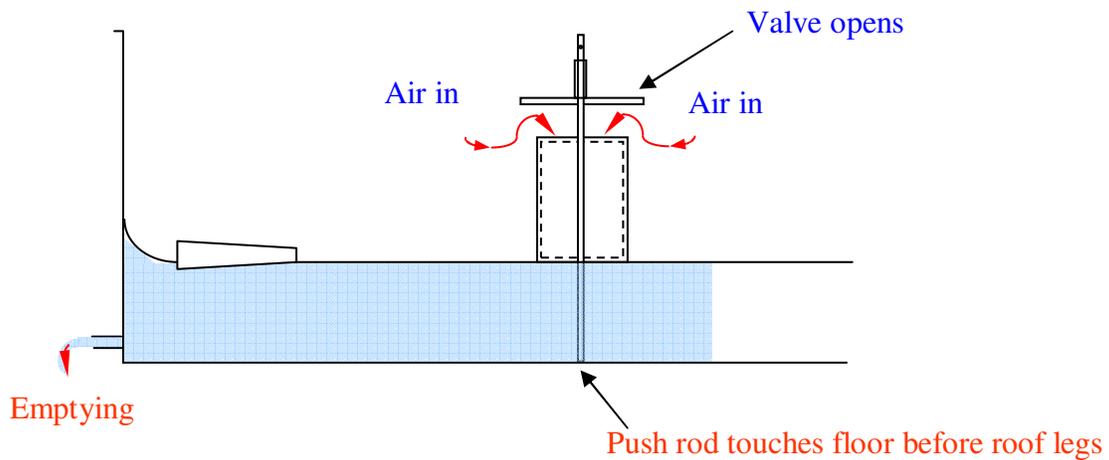


Figure 3.18 (b) Operating of Bleeder Vent during In-Breathing (Finishing)

In the case of filling in (Out-breathing), the roof is resting on the support legs and the valve is initially opened. The product start filling in, taking up the air space underneath the deck hence pushing the air/ vapour out through the valve. The valve will close after all the air beneath the roof had been expelled and the roof start floating on the product.

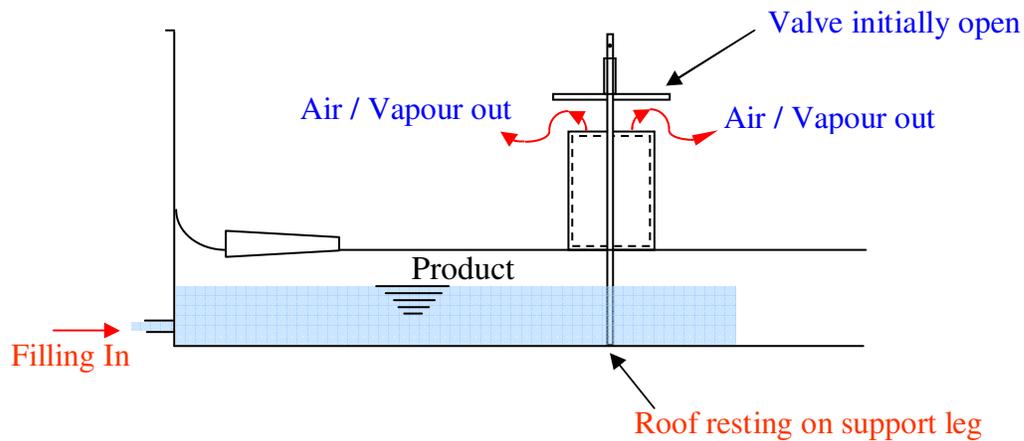


Figure 3.19 (a) Operating of Bleeder Vent during Out-Breathing (Starting)

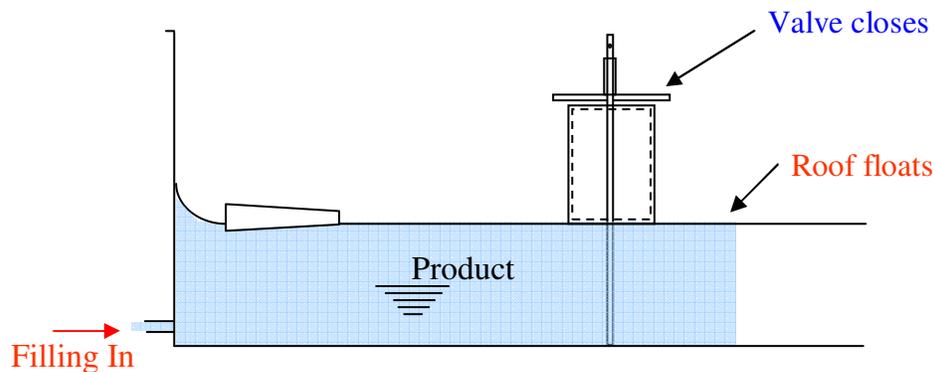


Figure 3.19 (b) Operating of Bleeder Vent during Out-Breathing (Finishing)

### 3.3.3.4.2 Bleeder Vent Design

The bleeder vent is to design accordance to API 2000 (1998) and sized up by using general flow equation. The requirements for normal venting capacity specified in API 2000 (1998) is that the total normal venting capacity shall be at least the sum of the venting requirements for oil movement and thermal effect [API 2000, 1998].

The design data for the venting design is as follow:

- Nominal Capacity = 24,000 m<sup>3</sup>
- Product Flash point = 67°C
- Design Filling Rate,  $V_i = 427$  m<sup>3</sup>/hr
- Design Emptying Rate,  $V_o = 1,100$  m<sup>3</sup>/hr

The venting capacity for both In-Breathing (Vacuum venting) and Out-Breathing (Pressure venting) has to be determined as per API 2000 (1998) requirement before the bleeder vent can be sized up. The maximum flow of the vacuum venting and pressure venting will be used to determine the minimum size and number of the bleeder vent.

i) The vacuum venting (In-Breathing)

The requirement for venting capacity for maximum liquid movement out of a tank will be 15.86 m<sup>3</sup>/h of free air for each 15.9 m<sup>3</sup>/h of maximum empty rate at any flash point [API 2000, 1998], which is

$$\text{Flow rate of free air for liquid movement, } V_{V1} = V_o / 15.9 * 15.86 = \underline{1,097.23 \text{ m}^3/\text{h}}$$

Thermal Breathing consideration is not requirement for the floating roof tank, therefore

Flow rate of free air for thermal breathing,  $V_{v2} = 0 \text{ m}^3/\text{h}$

The total vacuum flow required will be,

$$V_v = V_{v1} + V_{v2} = 1,097 \text{ m}^3/\text{h}$$

ii) The pressure venting (In-Breathing)

The requirement for venting capacity for maximum liquid movement out of a tank will be 17 m<sup>3</sup>/h of free air for each 15.9 m<sup>3</sup>/h (100 Barrel) of maximum filling rate [API 2000, 1998], which is

Flow rate of free air for liquid movement,  $V_{p1} = V_o / 15.9 * 17 = 457 \text{ m}^3/\text{h}$

Thermal Breathing consideration is not requirement for the floating roof tank, therefore

Flow rate of free air for thermal breathing,  $V_{v2} = 0 \text{ m}^3/\text{h}$

The total pressure flow required will be,

$$V_p = V_{p1} + V_{p2} = 457 \text{ m}^3/\text{h}$$

Therefore the maximum flow, Q is the vacuum flow which is 1,097 m<sup>3</sup>/h.

The below general flow equation below will be used,

$$Q = K.A\sqrt{2.g.H}$$

Where

$H =$  Head measures as pressure different, where

$$H = \frac{\Delta P}{\gamma} ; \quad \begin{array}{l} \Delta P = \text{Pressure different} \\ \gamma = \text{Specific weight of air} \end{array}$$

$g =$  gravity of acceleration, 9.81 m/s<sup>2</sup>

$A =$  Cross sectional area of vent

$K =$  Discharge Coefficient, 0.62 for circular

Re-arranging it to have it in term of area required, the equation becomes

$$A_{v\_req} = \frac{Q}{K} \sqrt{\frac{\gamma}{2 \cdot g \cdot \Delta P}}$$

Based on the equation, the minimum required venting area for the maximum flow capacity, Q was found to be 24,124 mm<sup>2</sup>. A vent size of 8" was pre-selected and the cross-sectional area available is 32, 251 mm<sup>2</sup>. There fore, the minimum number of bleeder vent required for the pre-selected size will be determine as

$$N_{vent} = \frac{A_{v\_req}}{A_v} = 1 \text{ no. of vent required (Minimum)}$$

However, total of 2 numbers will be installed in case one of it was blocked or not able function.

### 3.3.3.5 Roof Drain System

The roof drain system is to be installed in the floating roof tank to effectively drain the rain water from the floating roof without causing rain water to enter & contaminate the store product. The rainfall accumulated on the surface of the floating roof is drained to center sump which set into the lower point of the roof deck. The sump is then drained through a closed pipe work which operated with the tank. There is a non-return valve fitted to the outlet of the sump, which is to prevent the roof from being flooded with product in the event of a failure in the drain system. The drain pipe has to be removable for maintenance purposes, if required.

As the floating roof moves along with the product height, the basic requirement of the roof drain system has to be flexible to accommodate the roof movement. Figure 3.20 (a) and (b) show the drain system within the tank with the roof movement.

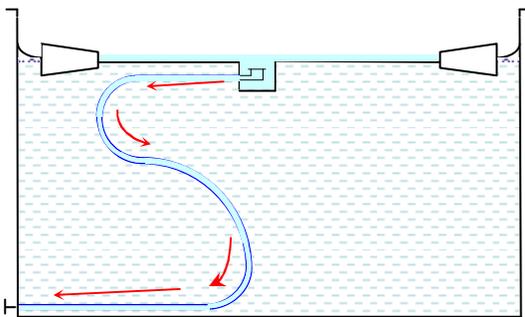


Figure 3.20 (a) Roof Drain with Roof Rise

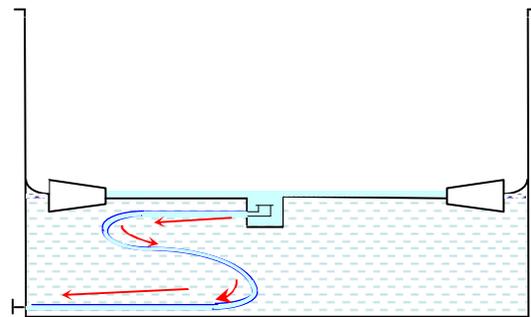


Figure 3.20 (b) Roof Drain with Roof Fall

There are several different drain systems available such as Articulated Piping System, Armoured Flexible Hose, Helical Flexible Hose or Pipe system. Rubber hose are strictly prohibited to be use in the oil tank and the two common systems used in the oil industry are the Articulated Piping System and Flexible Drain Pipe System. Therefore these two systems are selected for the study and evaluate their pros and cons, and then final selection of the system at the end of the evaluation.

### 3.3.3.5.1 Articulated Piping System

This drain system uses solid steel pipe with series of articulated knuckle joints such as flexible swing joint/ swivel joint. It also requires chain, shackles and pad eyes. Figure 3.21 shows the typical arrangement of an articulated piping system inside a floating roof tank.

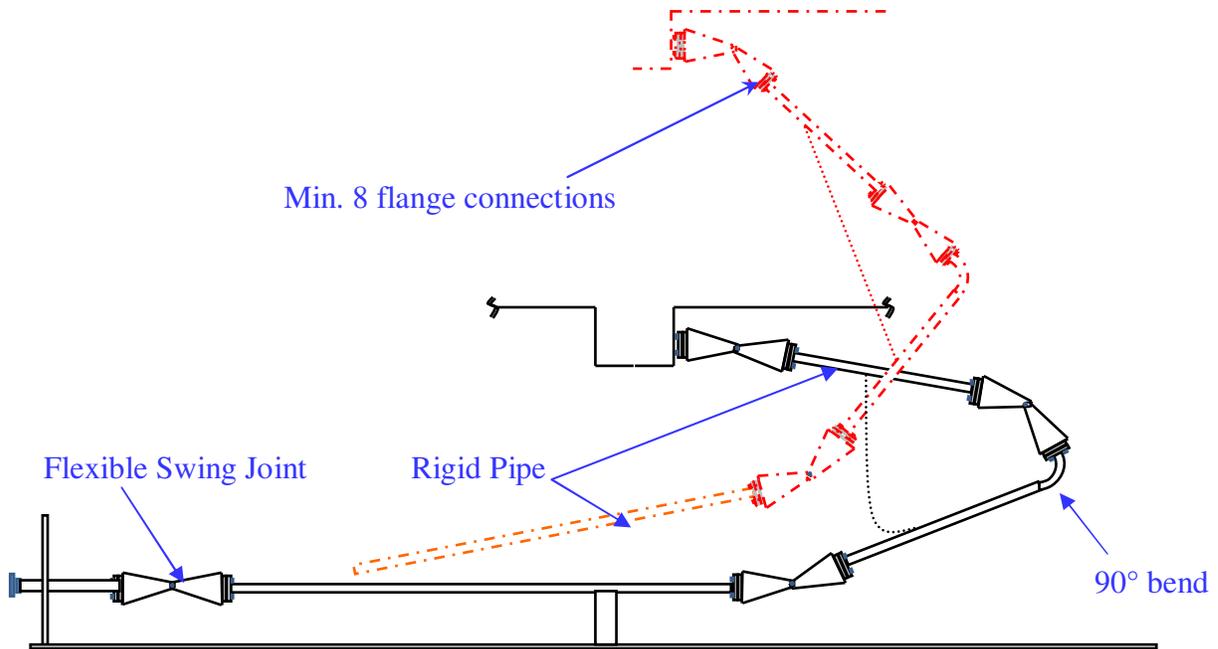
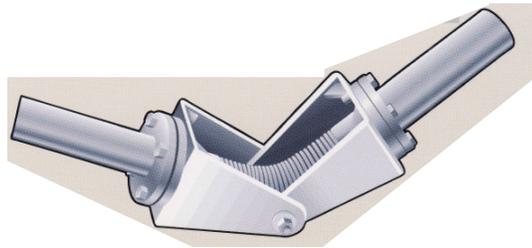


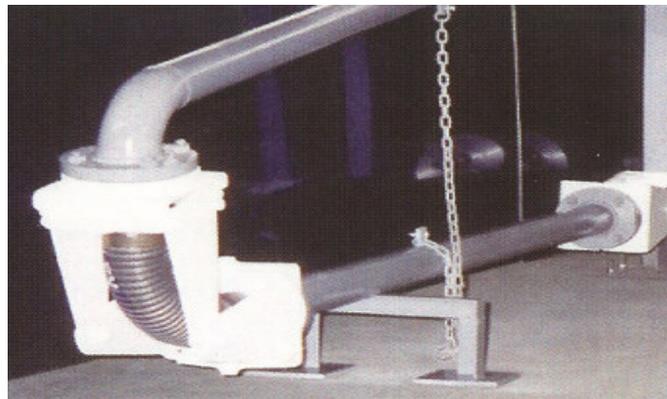
Figure 3.21 Articulated Drain Pipe System

The rigid pipes in the system caused the heavy weight to the system and may stress and distort the deck plate in the floating roof. There is also possibility of causing horizontal forces on to the roof which leads to wearing of the roof seal. The rigid pipes are connected to the swing/ swivel joint by flange connection, as can be seen in Figure 3.21, there will eight (8) connections, and two per each joints and each of these connections are potential to leak and also causes effect on the flow rate. There is a short 90° bend in the system and this short bend radius would able accumulate foreign material and blocked the drain.

Although this drain system is cheaper as compared to the Flexible Pipe System, but the installation of this system is considerably complicated and requires longer time which in turn causes a higher labour cost. The swing/ swivel joints and the flange connections are not easily accessible, which causes difficulties to perform any preventive maintenance. Figure 3.22 (a) and (b) show a diagram of a typical swing joint and its assembly. The actual articulated system and swing installed inside a floating tank can be seen in the Figure 1.26 and 1.27 in the Literature Review Chapter.



**Figure 3.22 (a) Typical Swing Joint in Articulated Drain Pipe System**



**Figure 3.22 (b) Swing Joint Assembly (Courtesy of WB)**

### 3.3.3.5.2 Flexible Drain Pipe System

The flexible drain system consist only single continuous pipe which expands and contracts with the rise and fall of the floating roof. Full length of the pipe is flexible and uniform without any joint. Figure 3.23 shows typical arrangement of the flexible drain system inside the floating roof tank.

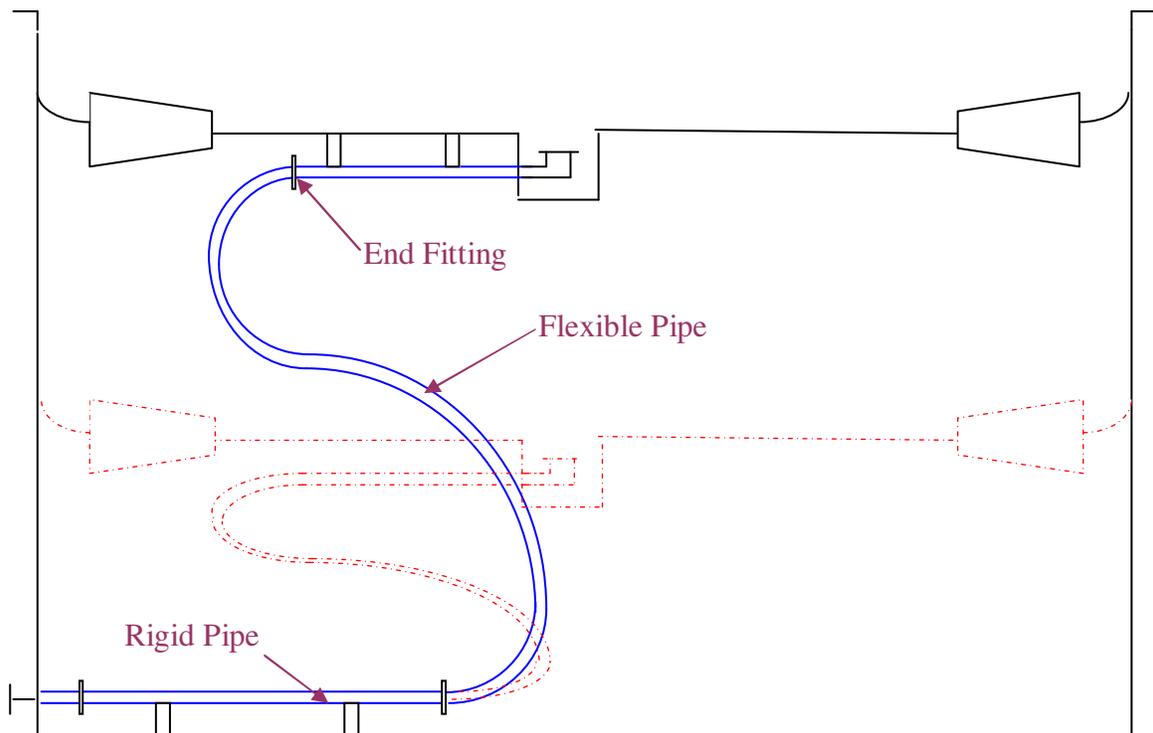
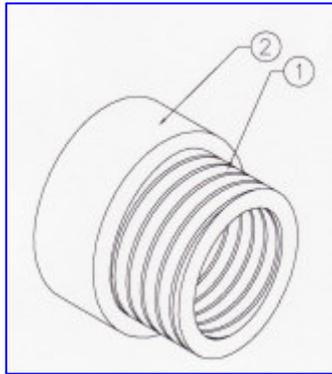


Figure 3.23 Flexible Drain Pipe System

There are no joints in the full length of the flexible pipe, the only connection is at the end fitting where it joins the flexible pipe to the top and bottom rigid pipe. The end fitting are integral part of the flexible pipe and hence the possibility of leakage is eliminated. The preventive maintenance is also eliminated. The flexible pipe is considerably much lighter than rigid pipe in the articulated pipe system and the arrangement is much simple, hence easy installation with lower installation and labour cost. However the material cost for the flexible is expensive. The flexible pipe in the system is known as COFLEXIP Flexible

pipe which the structure composed of an articulated stainless steel grade 304, spiral wound inner carcass covered by an outer extruded sheath of RILSAN Nylon 11. Figure 3.24 (a) shows the inner section of a COFLEXIP pipe and Figures 3.24 (b) shows the cut section of several different size of flexible pipe.



1. Inner interlocked Stainless Steel carcass (anti collapse) AISI 304.
2. External plastic sheath (RILSAN)

**Figure 3.24 (a) Inner Section of COFLEXIP Pipe** (Courtesy of *TECHNIP-COFLEXIP*)

The inner carcass is strong and flexes like hose but it will not kink or collapse. This inner carcass is designed to prevent leakage, instead it is the thick outer protective thermoplastic jacket made of RILSAN Nylon 11 which extruded over the inner carcass and form the water tight seal. Figure 3.25 shows the end fitting which is swaged around the drain pipe. A slip on Class 150 ANSI, rotating raised face flange is fitted behind the neck.



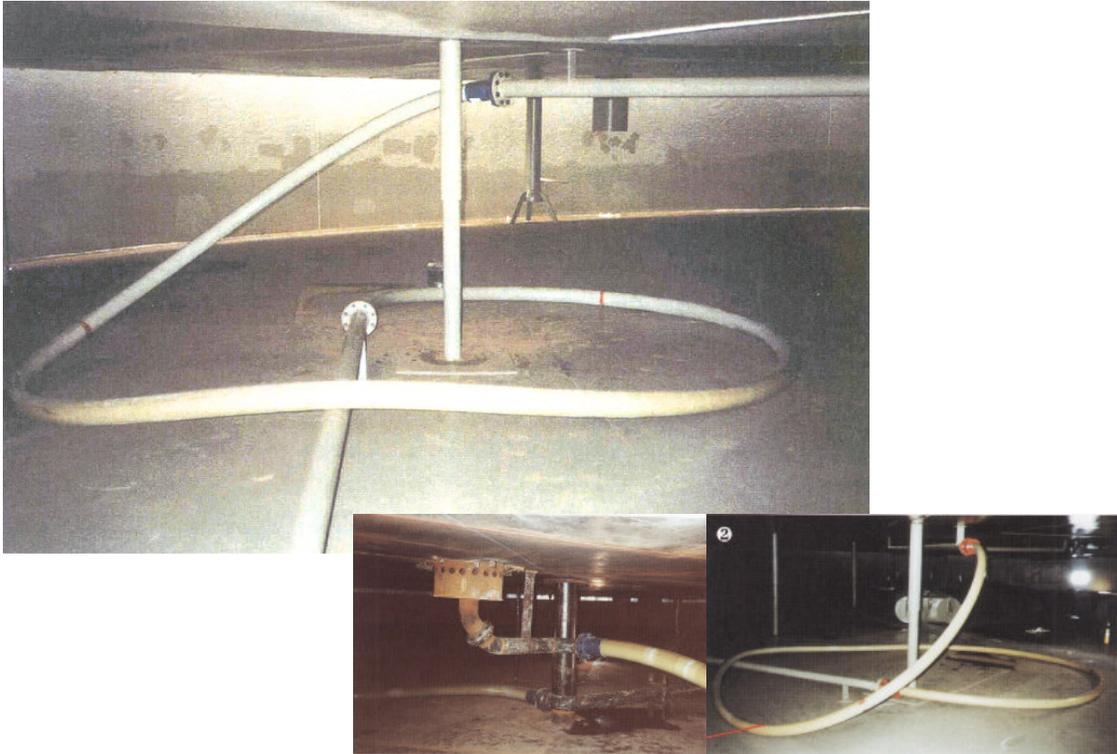
**Figure 3.24 (b) COFLEXIP Pipe of different size** (Courtesy of *TECHNIP-COFLEXIP*)



**Figure 3.25 End fitting of COFLEXIP Pipe** (Courtesy of *TECHNIP-COFLEXIP*)

### 3.3.3.5.3 Drain System Selection

It is obvious that the flexible drain pipe system has more advantage over the articulate piping system, except for the higher material cost. By looking into the cost saving of future maintenance and the service life, the flexible drain pipe is selected for my roof drain system. Figure 3.26 shows some example of actual flexible drain pipe system installed in different tank. It can be seen that the flexible pipe gives repeatable lay pattern which ensure no-fouling with the roof support leg.



**Figure 3.26 Flexible Drain Pipe System Installed in Different Tank**

### 3.3.3.5.4 Drain Pipe Design

The roof drain pipe is sized up using the general flow equation of  $Q = A \cdot V$ . The drain pipe size was pre-selected as 4" Schedule 80 and the minimum number of drain pipe required is to be determined. The drainage design data is as follow:

- Design Rain Fall, RH = 50 mm/hr
- Design Drainage Required = RH x deck area = 46.01 m<sup>3</sup>/hr
- Design Drain Pipe = 4" Sch 80 (O.D 101.6 x 8.56t)
- Drain Pipe Inside Diametr, d = 84.48 mm
- Roof Lowest Height = 1500 mm
- Drain outlet nozzle elevation, z = 225 mm

The total head equation is given as,

$$H = h + \frac{v^2}{2g};$$

And the total head loss of drain,

$$h = \frac{v^2}{2g} \times \left[ \frac{K_1 L_1'}{d} + \frac{K_2 L_2'}{d} \right];$$

Then, the total head equation becomes

$$H = \frac{v^2}{2g} \left[ \frac{K_1 L_1'}{d} + \frac{K_2 L_2'}{d} + 1 \right].$$

Re-arrange the equation, the flow velocity can be determined as follow:

$$V = \sqrt{\frac{2gH}{\left( \frac{K_1 L_1'}{d} + \frac{K_2 L_2'}{d} + 1 \right)}}$$

Where

$K$  = Flow Coefficient

- Rigid Pipe,  $K_1$  = 0.0168

- Flexible Pipe,  $K_2$  = 0.03

$L'$  = Total Equivalent Pipe Length

- Rigid Pipe,  $L_1'$

- Flexible Pipe,  $L_2'$

The equivalent pipe length of valve and fitting is determined accordance to Table 3.6 [NFPA 15, 2007]. The total equivalent pipe length will be the summation of the total equivalent length of the valve, fitting and the rigid and flexible pipe lengths.

Fittings and Valves	Fittings and Valves Expressed in Equivalent Feet (Meters) of Pipe													
	3½ in.		4 in.		5 in.		6 in.		8 in.		10 in.		12 in.	
	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m
45° Elbow	3	0.9	4	1.2	5	1.5	7	2.1	9	2.7	11	3.4	13	4.0
90° Standard elbow	8	2.4	10	3.1	12	3.7	14	4.3	18	5.5	22	6.7	27	8.2
90° Long turn elbow	5	1.5	6	1.8	8	2.4	9	2.7	13	4.0	16	4.9	18	5.5
Tee or cross (flow turned 90°)	17	5.2	20	6.1	25	7.6	30	9.2	35	10.7	50	15.3	60	18.3
Gate valve	1	0.3	2	0.6	2	0.6	3	0.9	4	1.2	5	1.5	6	1.8
Butterfly valve	—	—	12	3.7	9	2.7	10	3.1	12	3.7	19	5.8	21	6.4
Swing check*	19	5.8	22	6.7	27	8.2	32	9.8	45	13.7	55	16.8	65	19.8

**Table 3.6 Equivalent Pipe Length Chart [NFPA 15, p15]**

The flow velocity was calculated as 1.15m<sup>2</sup>/s, and substitute it into the flow equation of  $Q = A.V$ , the drainage flow rate for one drain pipe is found to be 23.3 m<sup>3</sup>/h. Therefore the minimum roof drain required are determined as

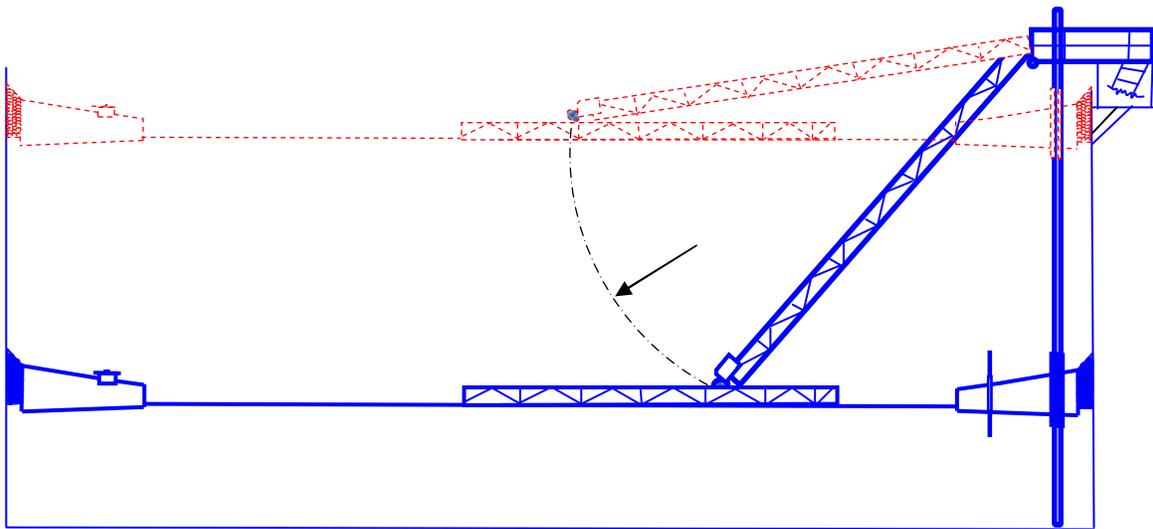
$$N_{\text{req}} = \frac{\text{Drainage Flow Rate Req.}}{\text{Actual Flow Rate}} = 1.97$$

Minimum two number of drain pipe with size of 4" schedule 80 will be used.

### 3.3.3.6 Rolling Ladder & Gauger Platform

The rolling ladder installed on the floating roof tank is to provide safe access onto the floating roof. The ladder consists of self-levelling treads and it slides along the track as the roof move up and down. The track and ladder length are matched to maximum and minimum roof height. The upper end of the ladder is attached to the gauger platform by hinged brackets and the lower end is provided with an axle with a wheel at each side of the ladder. The wheels run on a steel track mounted on a runway structure support off the roof.

The gauger platform is a small access area which overhangs on the shell, allowing instrumentation and guide pole to pass though. It also provides access for the maintenance personnel. Figure 3.27 shows the sketch of the rolling ladder and the gauger platform. Figure 3.28 shows some typical rolling ladder with the wheel and gauger platform installed in a floating roof tank.



**Figure 3.27 Sketch of Rolling Ladder and Gauger Platform in a Floating Roof Tank**

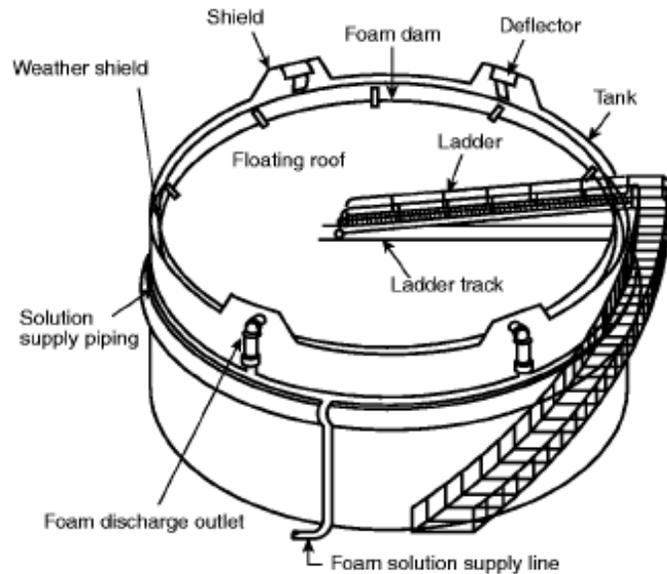


**Figure 3.28 Rolling Ladder and Gauger Platform Installed in a Floating Roof Tank**

### **3.3.3.7 Fire Fighting System and Foam Dam**

Fire on the floating roof tanks are common and it usually happened in the rim space where the vapour escaped, this was called as rim fires. The main cause of the floating roof rim fires is lighting. Most lighting ignited rim fires result from induced charges on the roof and not direct strikes. Fire fighting system is to be designed and installed on the floating roof to fight over and extinguishes the rim fire. There are several techniques available for the fire fighting and multiples foam chamber method is one it which will be discussed in detail here.

The multiple chamber method is which the foam is discharged by the foam chambers or foam pourer which mounted at equal spaced around tank periphery as shown in Figure 3.29. The system is to be designed accordance to NFPA-11 (Standard for low-medium- & high-expansion foam) [NFPA 11, 2005].



**Figure 3.29 General Arrangement of the Multiple Foam Chamber on the Floating Roof Tank [NFPA 11, P53]**

When the fires were detected, measures amounted of propriety foam making compound will be injected into the fire water system leading to the foam generating point of the tank. The foam generations are designed in such a way that to draw air into the mixture, causing the foam to expand as it was injected to the tank via pourer. The pourer inject the foam onto the internal surface of the extension of plate and hence onto the tank shell, causing it to flow down to the shell and collect and spread around the rim space. Figure 3.30 (a) show a typical arrangement of the fire protection for a floating roof tank and Figure 3.30 (b) show an actual foam dam installed on a floating roof tanks.

The foam is contained and concentrated within the rim space by a foam dam. Foam dam is a short vertical plate welded to pontoon at short distance from the seal. It's height shall be higher than upper tip of seal, allowing the whole seal area to be flooded with foam and extinguish fire effectively.

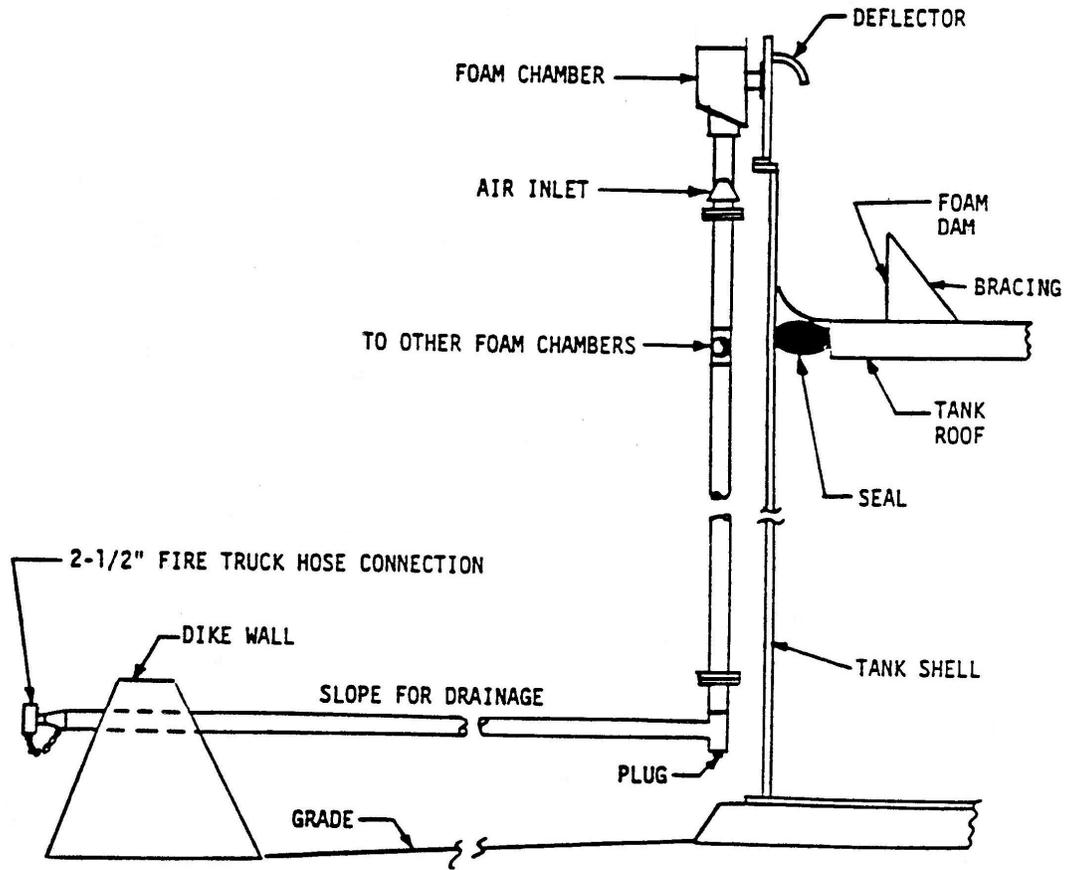


Figure 3.30 (a) Fire Protection for Floating Roof Tank



Figure 3.30 (b) Foam Chamber

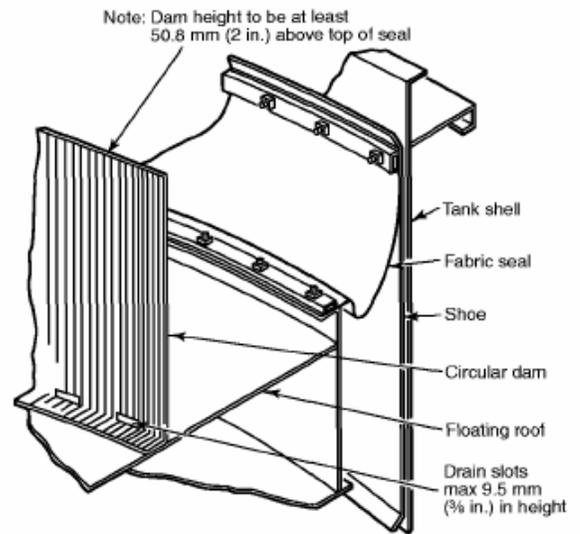


Figure 3.31 Typical Foam Dam [NFPA 11, p20]

## CHAPTER 4: TANK CONSTRUCTION

### 4.1 Introduction

Just as most of the construction task, welded vertical tanks can be erected satisfactorily in several ways, erector contractors normally have a particular method, which they have adopted as the result of experience, and have developed the erection technique most suitable for economical working and good workmanship by their field crews. Few erection methods are illustrated in Figure 4.1 (a) and (b). The method discussed here are simply the general method to give a basic idea on how a tank is built.

To build tanks which are of sound quality, good appearance and free from excessive buckles or distortion, correct welding sequences should be adhered to and adequate supervision provided.

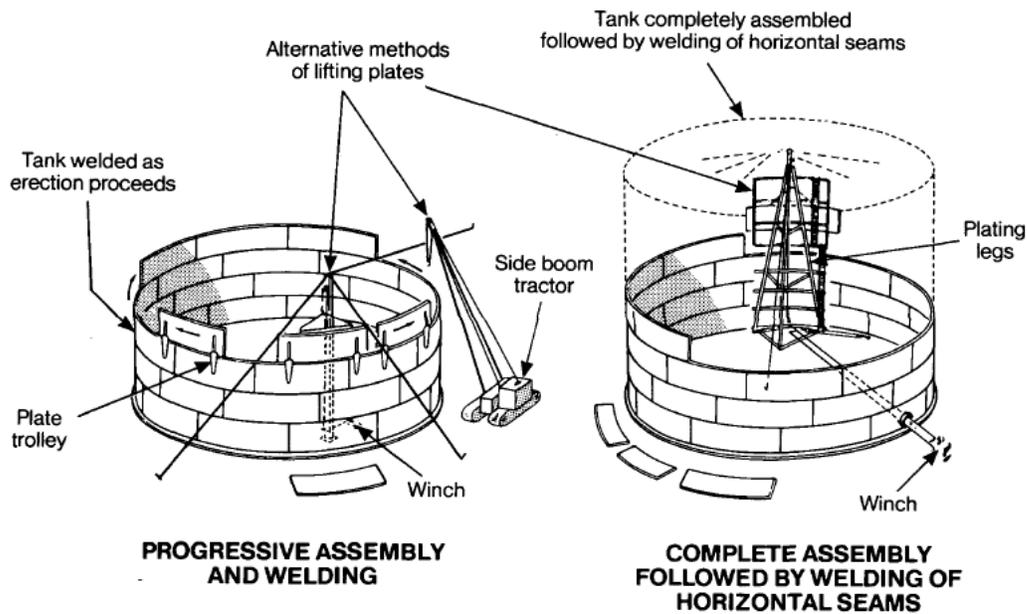


Figure 4.1 (a) Progressive Assembly & Welding and Complete Assembly Followed by Welding of Horizontal Seam Method for Welded Vertical Tank [PTS, 1986]

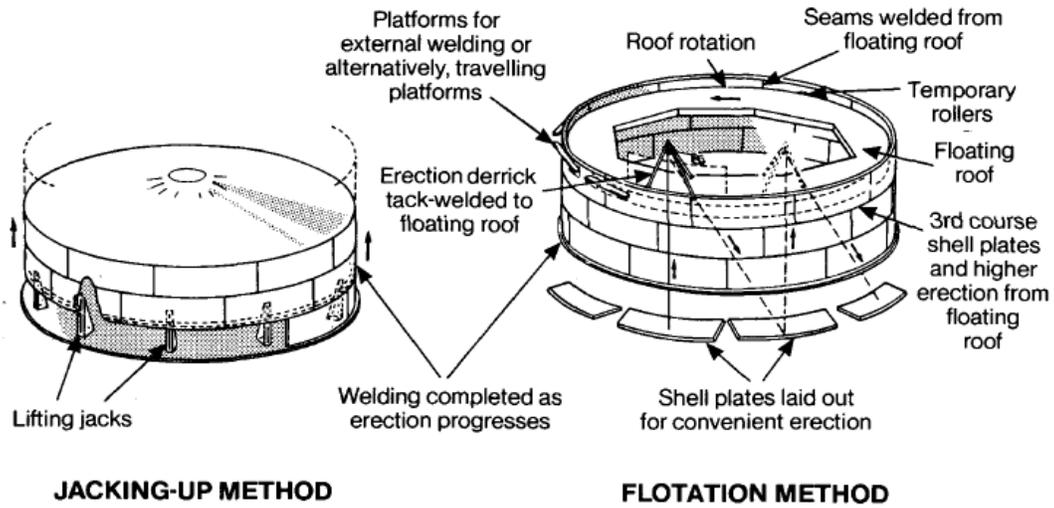


Figure 4.1 (b) Jacking-Up and Flotation Method for Welded Vertical Tank [PTS, 1986]

## 4.2 Foundation

Foundation has to be prepared well ahead before the tank construction start. A successful construction and operation of the tank relies on the tank being built on a firm foundation. API 650 (2007) Appendix B provides recommendations for design and construction of Foundation for above ground storage tanks. The construction and design will not be discussed in detail as our main concern is the tank itself.

One of the major parameter in designing and construction the tank foundation is the overturning moment and base shear force of the tank due to seismic and the anchor bolt arrangement and size. The foundation was built in a height of 300 mm from the ground level, anchor bolts are to be cast into the foundation as shown in Figure 4.2.



**Figure 4.2 Tank Foundation with anchor bolt installed**

### **4.3 Bottom Plate Placement**

When the tank foundation is done and ready for the tank erection, bottom plate will start laying on top of the foundation and welded in sequence. It is important to lay and weld the bottom plate in correct sequence to avoid any weld distortion.

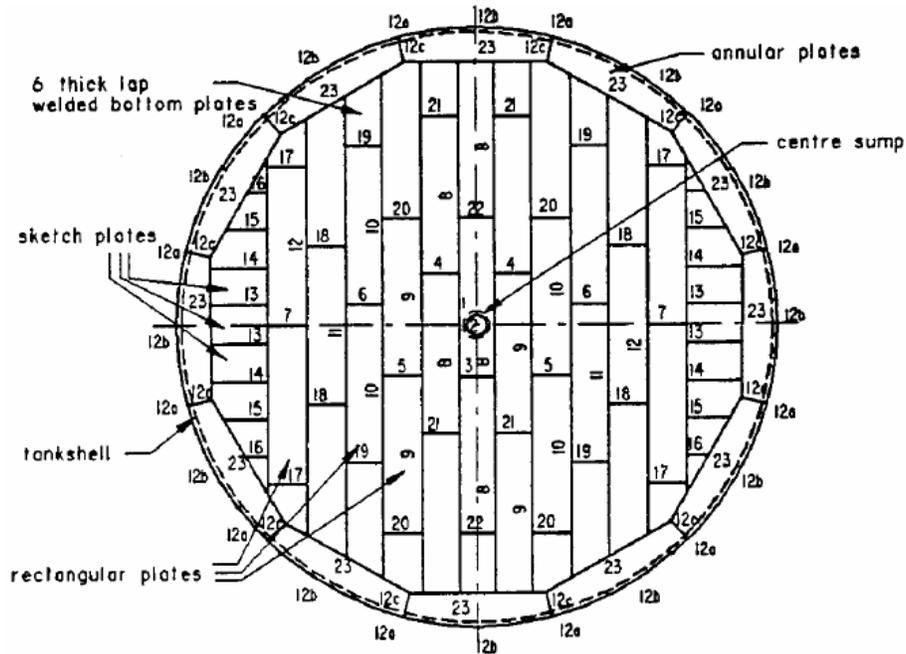


Figure 4.3 Bottom Plate Layout [PTS, 1986]

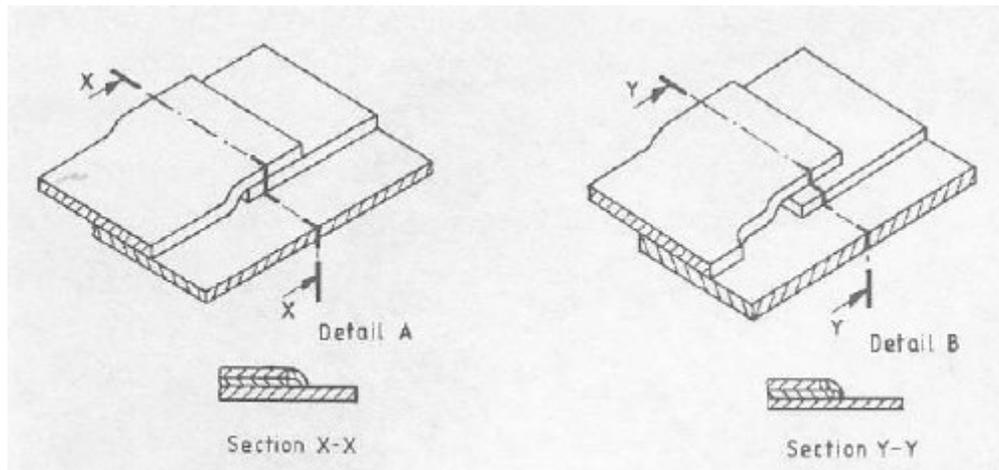
The welding sequence for bottom plate layout with annular plates, with reference to Figure 4.2 is as follow:

1. Lay plates and lightly tack –weld
2. Weld centre sump in position 1 and 2
3. Weld rectangular plates together commencing at centre, welding short seams first 3 to 11, seams between rows of plates shall be free of tack-welds before making final weld
4. Weld only outer part of radial seams of annular plates before erection of shell plates at 12a
5. After complete assembly and welding of lower shell courses, weld lower shell course to annular 12b for prevention of welding distortion.
6. Weld remaining part of radial weld of annulars at 12c
7. Weld rectangular and sketch plates together at 13 to 22 and finally to annulars at 23.

Figure 4.4 shows the actual bottom plate laying in top of the foundation on site, it shows that the bottom plates are laid in the lapping way. Figure 4.5 shows the detail of lap joints where three thicknesses occur.

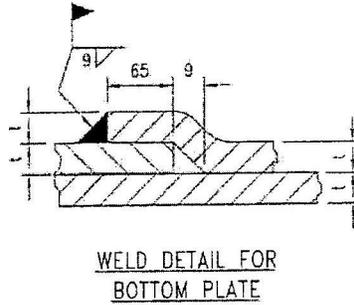


**Figure 4.4 Bottom Plate Laid on Foundation**



**Figure 4.5 Typical Cross Joint in Three Plate Lap**

Requirement in API 650 (2007) stated that the three-plate laps in the tank bottoms shall be at least 300 mm from each others, from the tank shell, from butt-welded annular plate joints, and from joints between annular plates and the bottom [API 650, 2007]. And the bottom plate need to be welded on the top side only, with continuous full-fillet weld on all seams as shown in the welding detail in Figure 4.6.



**Figure 4.6 Welding Detail for Bottom Plate**

#### **4.4 Shell Erection**

Shell plates will be erected when the bottom plates are done, the shell plates are held in place, tacked and completely welded. This will be done course by course, working upwards to the top curb angle. No course can be added as long as the previous course had not been entirely welded.

For the floating roof tank, Flotation Method as shown in Figure 4.1 (b) might be used, where upon completion of the bottom plating and erection of the two lower course of the tank, the floating roof is assembled on the tank bottom and completed. The tank is then filled with water and, using the floating roof as a working platform, the third and subsequent course are erected and welded, water being pumped in as each course is completed. However this method may only be used only at site where soil settlement is very limited. Refer to Chapter 5 for the soil settlement topic. Figure 4.7 show the complete erection of the first shell course and Figure 4.8 (a) and (b) show the erection of the upper shell course from the inside and outer of the tank respectively.



**Figure 4.7 Completed Erection of First Shell Course**



**Figure 4.8 (a) Erection of Upper Shell Course – Inside Tank**



**Figure 4.8 (b) Erection of Upper Shell Course – Outside Tank**

## **4.5 Tank Testing**

### **4.5.1 Tank Bottom Testing**

After welding of the bottom plates has been completed, all welds will be tested to ensure that the tank bottom is free from leaks. This can be done by using a vacuum box, which enables any leaks in the seams to be positively located by visual examination. The test is preferably be made as soon as possible after welding of the bottom but before any surface coating is applied. The bottom plates has to be tested before water is put into the tank for hydrostatic testing.

A typical vacuum box and pump is shown in Figure 4.9, where the vacuum box is fitted with a glass viewing panel on its top and has an open bottom, around which a continuous rubber seal and former are secured. The seal forms an airtight joint around the section of the weld to be tested when the box is pressed against the bottom plates. A partial vacuum can be created by means of a hand or motor-driven vacuum pump. A vacuum gauge is incorporated in the box which has two connections: one is the suction tap fitted with a non-return valve; the other is a vacuum release valve.

### **4.5.2 Tank Shell Testing**

The tank shells should be water tested/ hydrotested after completion of the wind girder. The tank will be filled up with water to its design level. The water test not only to ensure no leakage of the tank, it also tested the foundation for its capability of taking the filled tank load. Settlement will also be measured during the water testing.

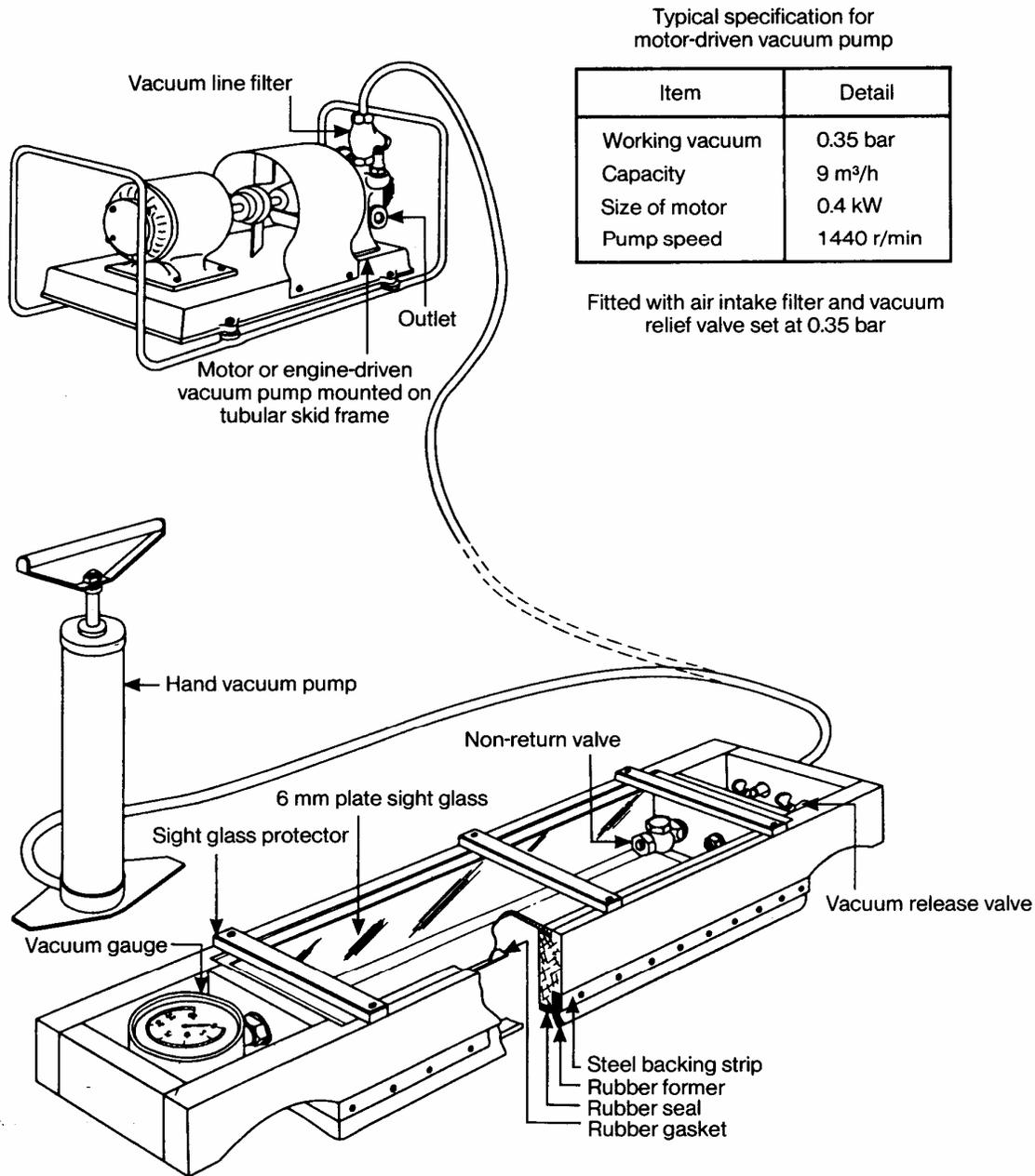


Figure 4.9 Vacuum Box and Pump [PTS, 1986]

### 4.5.3 Floating Roof Testing

The floating roof has to be liquid-tight in order for it to function safely and effectively, for all the weld seams and joints has to be liquid-tight, they will be inspected and tested in a more careful way, as follows [PTS, 1986]:

#### i) Centre Deck

The weld seams of the centre deck plates should be controlled on liquid-tightness by the vacuum box method or by the penetration oil/chalk method.

#### ii) Pontoon

Before the top plates of the pontoons are installed, the following seams have to be tested for tightness, using liquid dye- penetrants.

- The single-fillet welds on the upper surface of the pontoon bottom plates.
- The single-fillet welds between the bottom and the side walls of the pontoon.
- The single-fillet welds between the bulk heads and the bottom and side walls of the pontoon.
- The welds at the bottom comers of the bulk heads. These should be tested with particular care, as the bulk heads are mostly shaped at these points to clear the longitudinal weld between the bottom and the side walls. The gaps so formed in the comers must be effectively closed, as leaks in one compartment win allow oil to penetrate into the adjacent compartments.
- The longitudinal welds joining the centre deck to the pontoon.

### iii) Air Testing of Pontoon Compartments

After the completion of the liquid-tightness test for the floating roof pontoon, each individual pontoon compartment will also be checked by filling it with compressed air at a maximum pressure of 0.25 bar.

### iv) Roof Drain

The roof drain pipe systems for the floating roof will be tested with water to a pressure of 3.5 bar, and during the flotation test, the roof drains should be kept open and observed for leakage of tank contents into the drain lines.

## CHAPTER 5: SPECIAL CONSIDERATION

### 5.1 Design Consideration

#### 5.1.1 Design Consideration of Foundation

As mentioned earlier in Chapter 4 that providing adequate foundation is an important part of ensuring an economical and safe installation of storage tank. Poor foundation would threaten the integrity of the tank, no matter how good the tank design is. Uneven foundation settlement on floating tank is a special problem. The roof seal as discussed in Chapter 3.3.3.1 were designed to compensate for variation in tank diameter such as out-of-round, however in extreme condition; it will impair the roof seal efficiency and caused roof jamming. Therefore proper design of the foundation is essential to avoid the problem.

There are several types of soil settlement and only two of the common will has most effect on the floating roof will be discussed here.

##### i) Center-to-edge Settlement

Center-to-edge settlement results stretching of bottom plate, give rise to biaxial membrane tensile stresses. Excessive sagging causes the bottom-to-shell joint stress become excessive and eventually causes buckling. The maximum allowable sagging, see Figure 5.1, can be calculated as follow [PTS, 1986]:

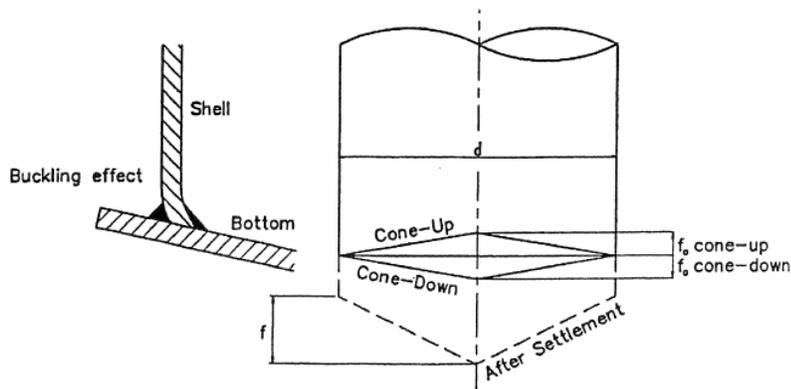
$$f = \frac{D}{100} \sqrt{\left(\frac{100f_o}{D}\right)^2 + 3.75}$$

Where

$f =$  Maximum allowable sag in the tank bottom, cm

$D =$  Diameter of tank, cm

$f_o =$  Deflection of bottom center, cm, in relation to bottom curb when the tank is erected (positive, zero or negative)



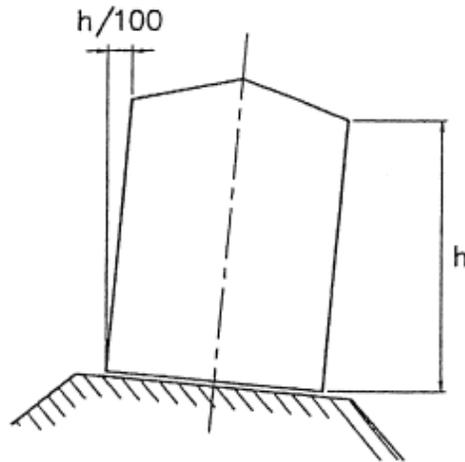
**Figure 5.1 Maximum Allowable Sag [EEMUA 2003, vol.1, p82]**

Some suggestion to the tank designer is that when large settlement is predicted, the bottom can be specified as cone-up bottom to minimise stresses in bottom plate and shell-to-bottom joint. The tank can also be lifted and re-pack the foundation before the settlement occurs.

## ii) Uneven Settlement around Circumference

Uneven or differential shell settlement around tank circumference would cause the tank tilted and significant out-of-roundness which result the floating roof to malfunction such as holding up, jamming, excessive emission of product vapours though seal gap and roof sinking.

As recommended in the EEMUA (2003), an in service settlement survey should be carried out preferably with the tank full, or nearly full, taking elevation reading at each survey point around the circumference. The minimum number of survey points would be diameter  $D$  in metres divided by 3.05, but should not be fewer than 8 and spacing between the survey points should not exceed a circumferential distance of 10 m. The maximum differential settlement between any two points at 10-metre intervals should not exceed 100 m, that is 1%. This limit was established to avoid severe localised stress increase in tank components. And the maximum out-of verticality at top of tank shell should not exceed  $1/100$  of tank height, see Figure 5.2. When limit is exceeded, re-leveling the tank and foundation should be considered. The limitation will have significant influence to the roof rim and rim seals design.



**Figure 5.2 Maximum Tolerances for Out-of Verticality of the Tank Shell [EEMUA 2003, vol.1, p81]**

### **5.1.2 Design Consideration on Tank Shell**

One of the major considerations on the tank shell is the local load acting on the tank shell wall. Shell wall are relative thin with respect to the large diameter, hence it has filmsy behavior. Any significant load acts on the shell wall has the potential causing buckling in the tank shell wall. Pipe support for the nozzle is one of the attachments which attached to the shell wall and exert a significant of load to it. The pipe supports shall be designed for the minimum load.

## 5.2 Construction Consideration

### 5.2.1 Nominal Diameter Versus Inside Diameter

Shell plate shall be aligned with inside diameter instead of the outside or nominal diameter during shell fit up. This is so that the shell wall would have a smooth surface for the roof seal to smoothly slide up and down without any jamming.

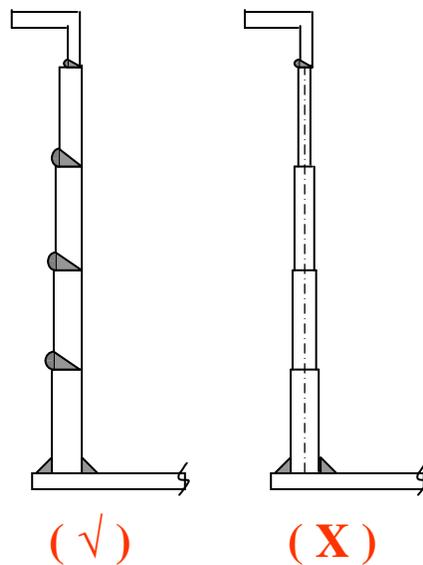


Figure 5.3 Alignment of Shell Plate for Welding

### 5.2.2 Plate Square-ness

Constructing a large tank requires plenty of steel plate, the plates are normally milled in rectangular shape, and the whole piece will use directly without any cutting except the edge preparation for the welding. However it is never the case, there are always plates come in an irregular shape where the square-ness were out. Therefore, extra length and width should be ordered for this irregularities and allowance to trim off the un-square side without affecting the over tank height and dimension.

### **5.2.3 Wind Damage**

The partial erected tank is very vulnerable to severe damage from the high wind load, temporary steel angle to be stitch welded to the shell acting as the temporary wind girder to resist buckling.

## **5.3 Testing Consideration**

### **5.3.1 Hydrotest/ Water Test**

Water is always an issue on construction site to fill up and test the huge tank. Some contractor who has limited knowledge on the tank and material properties, for cost saving purpose, they would use sea water as water medium to perform the water test. However sea water contains very high chlorine and it would cause corrosion to the tank. The materials selected were not designed for the sea water.

After the water test, never dewatering from the Manway or the clean out door, the tank venting were not designed for emptying in such big opening.

## CHAPTER 6: CONCLUSION

In completion of this thesis, I have understood what exactly a floating roof tank is all about. In the thesis, I had provided a basic design guideline on how to design a new floating roof tank with the special consideration upon the completion of the tank design.

Throughout the design process in the project, the design code - API 650 (2007) was strictly followed together with other standard such API 2000 (1998), NFPA 11 (2005), NFPA 15 (2007), Petronas Technical Specification (PTS) and many more. Several design spreadsheets was created to perform the tank design. The spreadsheet was created accordance to design codes and standard and the following designs were completed in the project:

- i) Shell Stress Analysis
- ii) Roof Stress Design
- iii) Selection of roof fitting
- iv) Sizing of roof fitting

In the shell stress analysis, by using the 1-foot method in API 650 (2007), the minimum shell wall thickness at the bottom course is 28 mm, and the thickness reduces accordingly with the liquid static head to 11 mm at the upper top course. The tank was found to be structurally stable without anchorage during the wind load; however it was structurally unstable for the seismic. Therefore anchorage is required.

In the roof stress design, the roof buoyancy was checked for the pontoon volume and the pontoon stresses was check and found structurally stable. Total 22 numbers of pontoon support legs and 30 numbers of deck support legs with size 4" pipe schedule 80 was designed. The bleeder vents were sized up to ( $\varnothing$  200 mm) 8" schedule standard pipe; minimum one number is required but total two were used as one will be designed for the standby purpose. Flexible drain pipe system was selected for the roof drain system and

minimum 2 numbers with size  $\text{Ø}$  100 mm is required. The Scissor Hanger Type in Metallic Mechanical Shoe Seal was selected for the primary seal in the roof seal system.

In the middle of the project, design verification was performed by using the finite element analysis (FEA) software - Abaqus, however after spending numerous of hour on the software, it was realized that the result given from the analysis is not helpful and essential. The design code used in my shell stress design had been well established; and had been used worldwide in the petrochemical industry over the past decades since 1919. It is not practical to verify their design in this project; however some derivation of formula were performed by studying and research of the basic stress theory. One example is the formulas for the minimum shell thickness in API 650 (2007) were derivate from the basic stress theory.

In completion of this dissertation, the operation of the floating tank was addressed thought the tank design. The tank can only be design only when the operation of the tank is well understood. Mechanical stress design for the tank and research of different type of roof fittings from different suppliers were carried out in the roof fitting design. The tank construction chapter had provided a basic understanding on how a floating tank is built and tested. Special consideration on the design and construction was also addressed. In summary, this dissertation had gives a basic guideline and summary to the tank designer on the Floating Roof Tank.

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**APPENDIX A**  
**(PROJECT SPECIFICATION)**

**ENG 4111/4112 Research Project**  
**PROJECT SPECIFICATION**

Project title: **Design, construction and operation of the floating roof tank in Turkmenistan**

Student: Kuan, Siew Yeng - 0050012450

Supervisor: Dr. Harry Ku  
*Co-Supervisor:* Dr. Talal Yusaf

*Sponsorship:*

**Project Synopsis:**

The project aims are to develop the design, construction and operation of a floating tank. The tank will be design accordance to its application. In this case, the tank is designed for petroleum industry use, that is to store the stabilised condensate which had been processed by other process equipments and systems before exporting to the SPM (Single- point mooring system) and offload to the Tanker. The project will consist of mechanical design of the tank. The operation of the tank will be defined; construction and erection methods of the tanks will be explained. Special design consideration will be addressed, various kinds of fitting and accessories will be designed and selected for the operation of the tank.

Mechanical design calculations will also be established using international code & Standard such as API Standard 650 – Welded Steel Tanks For Oil Storage (American Petroleum Institute) and client specifications. Inspection and testing of the tank will also be included. The main objective of the project is how to design a new floating roof tank.

**Timelines:**

1. Familiarization of equipment and literature reviews.

Begin : 9<sup>th</sup> March 2009  
Completion : 27<sup>th</sup> March 2009  
Approx. Hours : 60 hours

2. Define design & construction requirement, Code and client specification, special consideration before starting design.

Begin : 28<sup>th</sup> March 2009  
Completion : 10<sup>th</sup> April 2009  
Approx. Hours : 20 hours

3. Start design of Tank, select material, perform design calculation & drawing.

Begin : 11<sup>th</sup> April 2009  
Completion : 8<sup>th</sup> May 2009  
Approx. Hours : 50 hours

4. Design the fitting, mounting and accessories for the floating roof tank.

Begin : 9<sup>th</sup> May 2009  
Completion : 22<sup>th</sup> May 2009  
Approx. Hours : 30 hours

5. Tank construction & erection method & procedure, fabrication inspection.

Begin : 23<sup>th</sup> May 2009  
Completion : 26<sup>th</sup> June 2009  
Approx. Hours : 40 hours

6. Tank completion, final inspection & tank testing, tank operating.

Begin : 27<sup>th</sup> June 2009  
Completion : 3<sup>rd</sup> July 2009  
Approx. Hours : 20 hours

7. Drafting the thesis outline & present the design result, study report to supervisor.

Begin : 4<sup>th</sup> July 2009  
Completion : 10<sup>th</sup> July 2009  
Approx. Hours : 10 hours

8. 1<sup>st</sup> draft of thesis & preparation of power point slide for presentation. (Include residential school)

Begin : 11<sup>th</sup> July 2009  
Completion : 25<sup>th</sup> September 2009  
Approx. Hours : 80 hours

9. Final draft of thesis – incorporating comments and modification suggested by supervisor.

Begin : 26<sup>th</sup> September 2009  
Completion : 9<sup>th</sup> October 2009

Approx. Hours : 20 hours

9. Complete the thesis in requested format.

Begin : 10<sup>th</sup> October 2009  
Completion : 2<sup>nd</sup> November 2009  
Approx. Hours : 20 hours

AGREED:

 \_\_\_\_\_ (student)

\_\_\_\_\_ (Supervisor)

(Date) \_\_\_ / \_\_\_ / \_\_\_

**APPENDIX B**  
**(DESIGN CALCULATION)**

## STORAGE TANK DESIGN CALCULATION - API 650

### 1.0 DESIGN CODE & SPECIFICATION

DESIGN CODE : API 650 11th Edition

### 1.1 TANK

Item number : 7061T-3901  
 Roof ( Open/Close ) : Close  
 Type of roof ( Cone-roof / Dome-roof / Flat-roof / NA ) : Floating Roof

### 1.2 GEOMETRIC DATA

Inside diameter , Di ( corroded ) (@ 39,000 mm ) = 39,006 mm  
 Nominal diameter, Dn ( new ) ( based on 1st shell course ) = 39,028 mm  
 Nominal diameter, Dc ( corroded ) ( based on 1st shell course ) = 39,031 mm  
 Tank height (tan/tan), H = 20,700 mm  
 Specific gravity of operating liquid , S.G. (Actual) = 0.790  
 Specific gravity of operating liquid , S.G. (Design) = 1.00  
 Nominal capacity , V = 24736 m<sup>3</sup>  
 Maximum design liquid level, HL = 20,700 mm

### 1.3 PRESSURE & TEMPERATURE

Design pressure : Upper , Pu (Atmospheric) = 0.00 mbarg  
 : Lower , Pl = 0.00 mbarg Vac  
 Design temperature : Upper , Tu = 70 °C  
 : Lower , Tl = -17 °C

### 1.4 MATERIAL & MECHANICAL PROPERTIES

Component	Material	Tensile Stress St(N/mm <sup>2</sup> )	Yield Stress Sy(N/mm <sup>2</sup> )	Corrosion Allowance c.a.(mm)
<b>PLATE</b>				
Shell Plate ( Mat'l Code # 1 ) (bot)	A 516 GR. 65N	448.00	241.00	3.000
( Mat'l Code # 2 ) (top)	A 516 GR. 65N	448.00	241.00	3.000
Annular Plate	A 516 GR. 65N	448.00	241.00	3.000
Bottom Plate	A 516 GR. 65N	448.00	241.00	3.000
Roof Plate	A 516 GR. 65N	448.00	241.00	3.000
<b>STRUCTURE MEMBERS</b>				
Roof structure (rafter,bracing,etc )	A 516 GR. 65N	448.00	241.00	3.00
Top Curb Angle	A 516 GR. 65N	448.00	241.00	3.00
Intermediate Wind Girder	A 516 GR. 65N	448.00	241.00	3.00

**SHELL THICKNESS CALCULATION BY ONE-FOOT METHOD**

**2.0 SHELL DESIGN**

**2.1 GEOMETRIC DATA**

Plate size used : 2,440 mm  
 Shell plate min. width as per PTS 34.51.01.31 clause 6.3 : 1,500 mm

**2.2 MATERIAL & MECHANICAL PROPERTIES**

No	Material used	Specified min. tensile stress St (N/mm <sup>2</sup> )	Specified min. yield stress Sy (Nmm <sup>2</sup> )	Yield stress reduction fac ( App. M ) k	Max. allow design stress Sd (N/mm <sup>2</sup> )	Max. allow hydro.test stress St (N/mm <sup>2</sup> )	Corrosion allowance c.a (mm)
1	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
2	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
3	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
4	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
5	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
6	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
7	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
8	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
9	A 516 GR. 65N	448.00	241.00	1.000	160.67	180.75	3.00
10	-	-	-	-	-	-	-

**2.3 SPECIFIED MINIMUM SHELL THICKNESS**

Specification : API 650 11th Edition  
 Minimum thickness as per API 650 cl 5.6.1.1 = 8.00 mm  
 Minimum thickness as per PTS 34.51.01.31 = 11.00 mm

**2.4 SHELL THICKNESS CALCULATION BY ONE-FOOT METHOD ( CLAUSE 5.6.3.1 )**

SI METRIC UNIT :-

$$t_d = \frac{4.9D_c ([H+H_i] - 0.3) \cdot G}{S_d} + c.a$$

$$t_t = \frac{4.9D_n (H - 0.3)}{S_t}$$

Gravitational force = 9.81 m/s

t.min = Min. of t.design, t.hydro & min. thickness as per PTS.  
 tsc = Thicknes selected & used

**2.5 CALCULATION & RESULTS**

No.	Mat'l Code No.	Material	Width (mm)	Height (mm)	t.design (mm)	t.hydro. (mm)	t.min (mm)	tsc. (mm)	Result
1	1	A 516 GR. 65N	2,440	20,700	27.30	21.60	27.30	28.00	O.K.
2	1	A 516 GR. 65N	2,440	18,260	24.40	19.02	24.40	25.00	O.K.
3	1	A 516 GR. 65N	2,440	15,820	21.49	16.43	21.49	22.00	O.K.
4	1	A 516 GR. 65N	2,440	13,380	18.58	13.85	18.58	19.00	O.K.
5	1	A 516 GR. 65N	2,440	10,940	15.67	11.26	15.67	16.00	O.K.
6	1	A 516 GR. 65N	2,440	8,500	12.77	8.68	12.77	13.00	O.K.
7	1	A 516 GR. 65N	2,020	6,060	9.86	6.10	11.00	11.00	O.K.
8	1	A 516 GR. 65N	2,020	4,040	7.45	3.96	11.00	11.00	O.K.
9	1	A 516 GR. 65N	2,020	2,020	5.04	1.82	11.00	11.00	O.K.

2.6 MAXIMUM ALLOWABLE STRESS

No.	Height (mm)	t.min (mm)	tsc. (mm)	H' (mm)	H' max (mm)	$\Delta H$ (mm)	P'max N/m <sup>2</sup>	Pmax N/m <sup>2</sup>
1	20,700	27.30	28.00	20,700	21,306.77	606.77	5,952.41	5,952.41
2	18,260	24.40	25.00	18,260	18,786.53	526.53	5,165.29	5,165.29
3	15,820	21.49	22.00	15,820	16,266.30	446.30	4,378.18	4,378.18
4	13,380	18.58	19.00	13,380	13,746.06	366.06	3,591.06	3,591.06
5	10,940	15.67	16.00	10,940	11,225.82	285.82	2,803.94	2,803.94
6	8,500	12.77	13.00	8,500	8,705.59	205.59	2,016.82	2,016.82
7	6,060	11.00	11.00	6,060	7,025.43	965.43	9,470.87	2,016.82
8	4,040	11.00	11.00	4,040	7,025.43	2985.43	29,287.07	9,470.87
9	2,020	11.00	11.00	2,020	7,025.43	5005.43	49,103.27	29,287.07

H' = Effective liquid head at design pressure  
H' max = Max. liquid head for tsc.  
P'max = Max. allowable stress for tsc.  
Pmax = Max. allowable stress at shell course.

## BOTTOM & ANNULAR PLATE DESIGN

### 3.0 BOTTOM PLATE & ANNULAR PLATE DESIGN

Annular plate used ? ( yes/no )

: yes

#### BOTTOM PLATE

- (i) Minimum thickness as per API 650 Clause 5.4.1 = 6.00 mm  
Minimum thickness required (@ 3.00 mm c.a ) = 9.00 mm  
Therefore, use thickness of 9.00 mm (tb) is satisfactory.
- (ii) - = - mm
- (iii) Min. width of overlapping (cl. 5.1.3.5) = 25 mm
- (iv) Min. width of plate (cl. 5.4.1) = 1800 mm
- (v) - = 50 mm

#### ANNULAR PLATE

- (i) Nominal thickness of 1st shell course, tsc1 = 28.00 mm

Hydro. test stress in 1st shell course,

$$St = \frac{4.9Dn(H-0.3)}{tsc_1} = 139.33 \text{ N/mm}^2$$

where

- Dn = Nominal diameter, Dn ( new ) ( based on 1st shell course ) = 39.028 m  
H = Design liquid level = 20.700 m  
tsc<sub>1</sub> = Nominal thickness of 1st shell course = 28.000 mm

Annular plate thickness ( As per Table 5-1a )

- Minimum thickness required (@ 3.00 mm c.a. ) = 9.00 mm  
Therefore, use thickness of 16.00 mm (ta) is satisfactory.

- (ii) Min. shell-to-bottom fillet welds size (cl. 5.1.5.7) = 13.00 mm  
(iii) Min. width projected inside of shell to edge of overlapping (cl. 5.5.2) = 600 mm

- (iv) Min. radial width of annular plate (cl. 5.5.2)

$$La = \frac{215 ta}{(HL \cdot SG)^{0.5}} = 756.09 \text{ mm}$$

where

- ta = Annular plate thickness = 16.000 mm  
HL = Maximum design liquid level = 20.70 m  
SG = Design specific gravity = 1.00

- (v) Min. width projected outside of shell ( cl. 5.5.2) = 50 mm

## ROOF TO SHELL JUNCTION CALCULATION

### 4.1 DESIGN OF OPEN ROOF TANK - TOP STIFFENER RING

#### 4.1.1 TOP CURB ANGLE

If the top wind girder is located 600 mm below top of the tank, top curb angle shall be provided.

Location of top wind girders from top of tank, L = 1000 mm

Since L is > 600mm from top of tank, top curb angle is **required.**

#### MINIMUM REQUIREMENT

Minimum required size as per API 650 clause 5.9.3.2 = 76 x 76 x 6.4

Section modulus, Z min = 8380 mm<sup>3</sup>

#### MEMBER SIZE USED FOR TOP CURB ANGLE

Actual size for top curb angle = 75 x 75 x 10

Section modulus, Z<sub>a</sub> = 13500 mm<sup>3</sup>

Since Z<sub>a</sub> > Z<sub>min</sub>, therefore the angle size selected is **satisfactory.**

#### 4.1.2 TOP WIND GIRDER

The required minimum section modulus of the stiffening ring shall be as follows:-

$$Z = \frac{D_c^2 \cdot H_2}{17} \left( \frac{V}{190} \right)^2 = 1007 \text{ cm}^3 = 1,007,140 \text{ mm}^3$$

where

D<sub>c</sub> = Nominal Tank Diameter = 39.031 m  
 H<sub>2</sub> = Height of tank shell = 20.7 m  
 V = Wind Velocity = 140.00 km/hr

#### MEMBER SIZE USED FOR TOP WIND GIRDER

Available section modulus

Fabricated Tee- Girder : T 825 x 250 x 8 x 10

Web plate length, L<sub>2</sub> = 825 mm

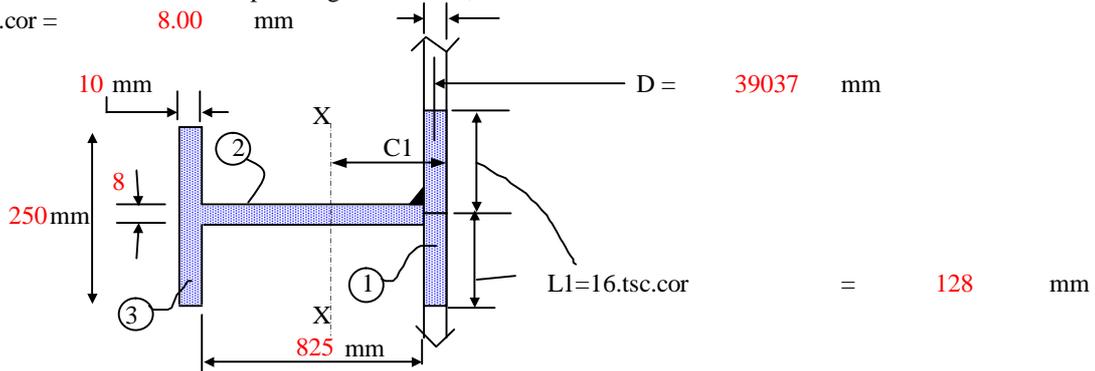
Toe plate length, L<sub>3</sub> = 250 mm

Web plate thk, t<sub>2</sub> = 8 mm

Toe plate thk, t<sub>3</sub> = 10 mm

Min. shell thickness where top wind girder located, t<sub>sc.cor</sub> = 8.00 mm

t<sub>sc.cor</sub> = 8.00 mm



	A (mm <sup>2</sup> )	Y (mm)	AY (mm <sup>3</sup> )	h (mm)	A.h <sup>2</sup> (mm <sup>4</sup> )	I = (bd <sup>3</sup> )/12 (mm <sup>4</sup> )
1	2048	4.00	8192	433.61141	385062615	10,923
2	6600	420.5	2775300	17.1114101	1932482.35	374343750
3	2,500	838.00	2,095,000	400.39	400,777,557	20,833
<b>TOTAL</b>	<b>11,148</b>		<b>4,878,492</b>		<b>787,772,655</b>	<b>374,375,506</b>

Neutral axis of combined section, C1 = 438 mm

Moment of inertia of section, I<sub>x-x</sub> = 1,162,148,161 mm<sup>4</sup>

Section modulus available, Z<sub>a</sub> = 2,655,662 mm<sup>3</sup>

Since Z<sub>a</sub> > Z<sub>min</sub>, therefore the angle size selected is **satisfactory.**

**INTERMEDIATE WIND GIRDERS CALCULATION**

**5.0 INTERMEDIATE WIND GIRDERS DESIGN**

**5.1 MAXIMUM HEIGHT OF THE UNSTIFFENED SHELL ( CLAUSE 5.9.7.1 )**

SI METRIC UNIT :-

$$H_1 = (9.47 \text{ ts.cor}) \sqrt{\left(\frac{\text{ts.cor}}{\text{Dc}}\right)^3} \times \left(\frac{190}{V}\right)^2$$

= 9.182 m  
= 9182 mm

where      ts.cor = Top shell course thickness      = 8.00 mm  
               Dc = Nominal tank diameter                = 39.03 m  
               V = Wind design speed                        = 140.00 km/hr

**5.2 LOCATION OF INTERMEDIATE WIND GIRDERS**

Shell course	Shell thickness tsc.cor (mm)	Actual width W (mm)	Transposed width Wtr (mm)
1	25.00	2,440	141
2	22.00	2,440	195
3	19.00	2,440	281
4	16.00	2,440	431
5	13.00	2,440	725
6	10.00	2,440	1,397
7	8.00	2,020	2,020
8	8.00	2,020	2,020
9	8.00	2,020	2,020
10	-	-	-
11	-	-	-
12	-	-	-
13	-	-	-
14	-	-	-
15	-	-	-
Height of transformed shell, H2 =			9,230 mm

Since H1 < H2, therefore the intermediate wind girder is/are **required**

Minimum number of intermediate wind girders required,  
 = **1**

Location of intermediate wind girders from top of tank,

L1 = **4615 mm**  
 L2 = - mm  
 L3 = - mm  
 L4 = - mm  
 L5 = - mm

5.3 SIZE OF INTERMEDIATE WIND GIRDERS

(a) Required minimum section modulus of intermediate wind girder ( clause 5.9.7.6 )

SI METRIC UNIT :-

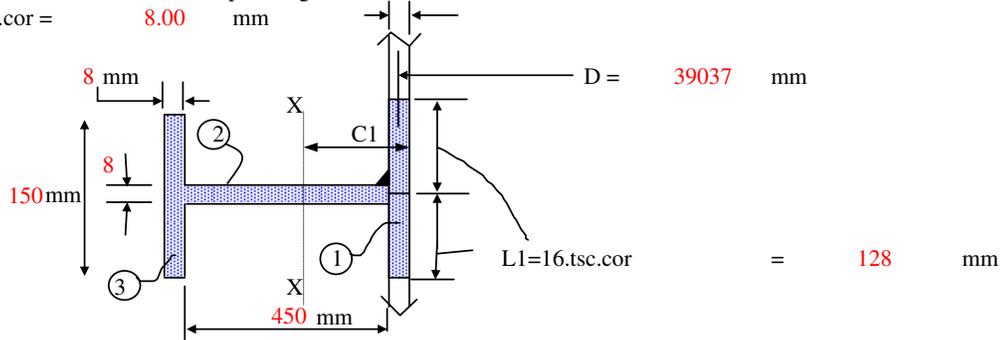
$$Z_{min} = \frac{Dc^2 \cdot H1}{17} \left( \frac{V}{190} \right)^2 = 225.812 \text{ cm}^3 = 225,812.032 \text{ mm}^3$$

where

Dc = Nominal tank diameter = 39.031 m  
 H1 = Vertical dist. between inter. wind girder & top angle = 4.615 m  
 V = Wind design speed = 140.40 km/hr

(b) Available section modulus for intermediate wind girder

Fabricated Tee- Girder : T 405 x 150  
 Web plate length, L2 = 450 mm  
 Toe plate length, L3 = 150 mm  
 Web plate thk, t2 = 8 mm  
 Toe plate thk, t3 = 8 mm  
 Min. shell thickness where top wind girder located, tsc.cor = 8.00 mm



	A (mm <sup>2</sup> )	Y (mm)	AY (mm <sup>3</sup> )	h (mm)	A.h <sup>2</sup> (mm <sup>4</sup> )	I = (bd <sup>3</sup> )/12 (mm <sup>4</sup> )
1	2048	4.00	8192	200.642523	82447200.6	10,923
2	3600	233	838800	28.3574766	2894927.33	60750000
3	1,200	462.00	554,400	257.36	79,479,445	6,400
TOTAL	6,848		1,401,392		164,821,573	60,767,323

Neutral axis of combined section, C1 = 205 mm  
 Moment of inertia of section , Ix-x = 225,588,896 mm<sup>4</sup>  
 Section modulus available, Za = 863,143 mm<sup>3</sup>  
 Since Za > Zmin , therefore the angle size selected is satisfactory.

## 6.0 WIND LOAD CALCULATION (OVERTURNING STABILITY)

### 6.1 WIND DESIGN CALCULATION

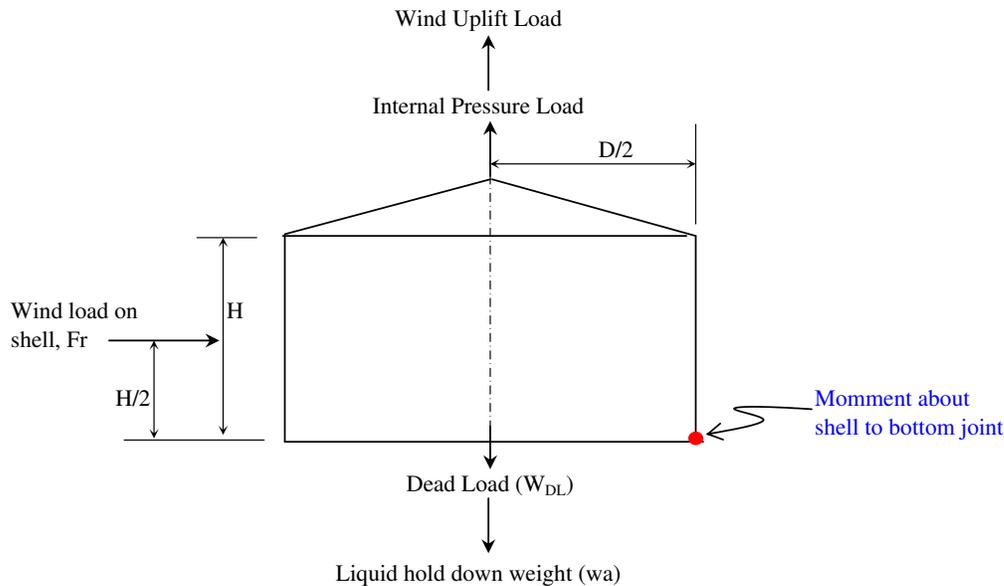
Internal design pressure, $P_i$ ( @	0.0	mbarg. )	=	0 N/mm <sup>2</sup>
Insulation thickness, $t_i$			=	75 mm
Nominal diameter of tank, $D$			=	39,000 mm
Tank height , $H_s$			=	20,700 mm
Roof slope, $\beta^\circ$			=	0.000 °
Roof height, $H_r$			=	0 mm
Height from tank bottom to shell centre, $L_s$			=	10,350 mm
Height from tank bottom to roof centre, $L_r$			=	20,700 mm
Min. depth of product (always present in tank) , $H_w$			=	0 mm
Weight of tank, $W_t$ (corroded condition) ( @	550,045	kg )	=	5,395,939 N
Weight of product (always present in tank) , $W_w$			=	0 N
Weight of shell + top angle (corroded) , $W_{DL}$ ( @	327,512	kg )	=	3,212,898 N

### 6.2 WIND FORCE CALCULATION

As per API 650 clause 5.2.1(j), the wind pressure are as follows:-

Wind pressure on conical surfaces, $w_r$ ( @	30.00	psf )	=	0.0014369 N/mm <sup>2</sup>
Wind pressure on cylindrical surfaces, $w_s$ ( @	18.00	psf )	=	0.0008621 N/mm <sup>2</sup>
Wind correction factor, $k_w$ ( $= V / 190$ ) <sup>2</sup>			=	1.00
Projected area of roof, $A_r$ ( $= 0.5.k.Do.Hr$ )			=	0 mm <sup>2</sup>
Projected area of shell, $A_s$ ( $= k.Do.H_s$ )			=	811,564,200 mm <sup>2</sup>
Total wind load exerted on roof, $F_r$ ( $= w_r.k_w.A_r$ )			=	0 N
Total wind load exerted on shell, $F_s$ ( $= w_s.k_w.A_s$ )			=	699,681 N
Total wind moment on tank, $M_w$ ( $= F_r.L_r + F_s.L_s$ )			=	7,241,700,964 Nmm

### 6.3 OVERTURNING STABILITY AGAINST WIND LOADING



For tank to be structurally stable without anchorage, the following uplift criteria shall satisfy:

Criteria 1:  $0.6 M_w + M_{pi} < M_{DL} / 1.5$

Criteria 2:  $M_w + 0.4 M_{pi} < (M_{DL} + M_F) / 2$

where:

$M_{pi}$  = Moment about the shell-to-bottom joint from design internal pressure  
 = Uplift thrust on roof due to internal pressure x 1/2 tank diameter  
 =  $(1/4 \pi \cdot D^2 \cdot P_i) \cdot 1/2 \cdot D$  = 0 Nmm  
 $M_w$  = Overturning moment about the shell-to-bottom joint from horizontal

	plus vertical wind pressure		
	Total wind moment on tank, ( = Fr.Lr + Fs.Ls )	=	7,241,700,964 Nmm
$M_{DL}$	Moment about the shell-to-bottom joint from the weight of the shell and the roof supported by the shell.		
	$0.5 \cdot D \cdot W_{DL}$	=	62,651,502,376 Nmm
	<i>Weight of roof = 0, since it is floating on liquid</i>		
$M_F$	Moment about the shell-to-bottom joint from liquid weight (wa)	=	153,419,379,181 Nmm
	$\frac{(wa \cdot \pi D) \cdot D}{1000 \cdot 2}$		
wa =	Weight of liquid = $59 \cdot tb \cdot \sqrt{F_{by} \cdot H}$	=	64,214.21 N/m
H =	Design liquid height	=	19.2 m
tb =	Thickness of Bottom plate under the shell	=	16 mm
F <sub>by</sub> =	Minimum specified yield stress of the bottom plate under the shell	=	241 N/mm <sup>2</sup>

**FOR CRITERIA 1**       $0.6 M_w + M_{pi} < M_{DL} / 1.5$

$0.6 M_w + M_{pi}$	=	4,345,020,578 Nmm
$M_{DL} / 1.5$	=	41,767,668,251 Nmm

**FOR CRITERIA 2**       $M_w + 0.4 M_{pi} < (M_{DL} + M_F) / 2$

$M_w + 0.4 M_{pi}$	=	7,241,700,964 Nmm
$(M_{DL} + M_F) / 2$	=	108,035,440,779 Nmm

Since,  
 $0.6 M_w + M_{pi} < M_{DL} / 1.5$ , and  
 $M_w + 0.4 M_{pi} < 1/2 (M_{DL} + M_F)$

The tank anchorage is **NOT REQUIRED**.

**7.0 SEISMIC FORCE CALCULATION**

**7.1 SEISMIC LOADS DESIGN**

**7.1.1 GEOMETRIC DATA**

Seismic peak ground acceleration, Sp	=	0.3 g
Importance factor, I	=	1.50
Site Class	=	D
Seismic Use Group, SUG	=	III
Nominal diameter of tank, D	=	39,031 mm
Total height of tank shell, Ht	=	20,700 mm
Ht.from bottom shell to COG of shell,Xs	=	10,350 mm
Maximum design liquid level, H	=	20,700 mm
Ht.from bottom shell to COG of roof,Xr	=	0 mm
Design specific gravity of liquid, G	=	1
Total weight of tank shell, Ws	( @ 352,948 kg )	= 3,462,418 N
Total weight of tank roof, Wr	( @ 0 kg )	= 0 N
Total weight of tank contents, Wp	( @ 24,728,026 kg )	= 242,581,931 N
Total weight of tank bottom, Wf	( @ 84,961 kg )	= 833,471 N

*Note: The total weight of the tank roof will be added to the weight of tank content, since the roof is floating on the liquid.*

**7.1.2 DESIGN SPECTRAL RESPONSE ACCELERATIONS**

Impulsive spectral acceleration parameter, Ai

$$A_i = 2.5 Q F_a S_o \frac{I}{R_{wi}} = 0.34$$

Convective spectral acceleration parameter, Ac

**When  $T_c \leq T_L$**

$$A_c = 2.5 K Q F_a S_o \left( \frac{T_s}{T_c} \right) \left( \frac{I}{R_{wc}} \right) \leq A_i = -$$

**When  $T_c > T_L$**

$$A_c = 2.5 K Q F_a S_o \left( \frac{T_s \cdot T_L}{T_c^2} \right) \left( \frac{I}{R_{wc}} \right) \leq A_i = 0.063298299$$

where

Q =	Scaling factor	=	1
K =	Coefficient to adjust the spectral damping from 5% - 0.5%	=	1.5
Fa =	Acceleration based site coefficient as per Table E-1	=	1.2
Fv =	Velocity-based site coefficient as per Table E-2	=	1.65
So =	Substitution for seismic peak ground acceleration Sp	=	0.3
Rwi =	Force reduction coefficient for impulsive mode as per Table E-4	=	4
Rwc =	Force reduction coefficient for convective mode as per Table E-4	=	2
T <sub>L</sub> =	Regional dependent transition period for longer period ground motion	=	4 s
T <sub>c</sub> =	First mode sloshing wave period for convective mode	=	6.63 s
T <sub>s</sub> =	Fv. S1/ Fa. Ss	=	0.69

### 7.1.3 CONVECTIVE (SLOSHING) PERIOD

The first mode sloshing wave period,

$$T_c = 1.8 K_s \sqrt{D} = 6.63 \text{ s}$$

where,

$K_s$  = sloshing period coefficient

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68 H}{D}\right)}} = 0.59$$

$$T_s = \frac{F_v \cdot S_1}{F_a \cdot S_s} = 0.69$$

where,

$F_a$  = Acceleration based site coefficient (at 0.2 sec period) as per Table E-1 = 1.2

$F_v$  = Velocity-based site coefficient (at 1 sec. period) as per Table E-2 = 1.6500

$S_1$  = Maximum considered earthquake, 5% damped, spectral response acceleration parameter at the period of one second, %g

$S_s$  = Maximum considered earthquake, 5% damped, spectral response acceleration parameter at short period of 0.2 second, %g

For regions outside USA, sites not defined by ASCE 7 method,

$$S_1 = 1.25 S_p = 0.375$$

$$S_s = 2.5 S_p = 0.75$$

Since  $T_c > T_L$ , the convective spectral acceleration parameter  $A_c$  = 0.06  
and the impulsive spectral acceleration parameter  $A_i$  = 0.34

## 7.2 OVERTURNING STABILITY AGAINST SEISMIC LOADING

### 7.2.1 EFFECTIVE MASS OF TANK CONTENTS

Effective impulsive portion of the liquid weight,

For  $D/H \geq 1.333$ ,

$$W_i = \left[ \frac{\tanh(0.866 \cdot D/H)}{0.866 \cdot D/H} \right] \cdot W_p = 137,636,499.10 \text{ N}$$

For  $D/H < 1.333$ ,

$$W_i = \left[ 1.0 - 0.218 \frac{D}{H} \right] \cdot W_p = - \text{ N}$$

Since  $D/H > 1.333$ , effective impulsive portion of the liquid weight,  $W_i$  = 137,636,499.10 N

Effective convective weight,

$$W_c = 0.230 \frac{D}{H} \tanh\left(\frac{3.67H}{D}\right) \cdot W_p = 100,998,137.14 \text{ N}$$

### 7.2.2 CENTER OF ACTION FOR EFFECTIVE LATERAL FORCES

The height from the bottom of the Tank Shell to the center of action of the lateral seismic forces related to the **impulsive liquid force** for ringwall moment,

For  $D/H \geq 1.333$ ,

$$X_i = 0.375H = 7762.5 \text{ mm}$$

For  $D/H < 1.333$ ,

$$X_i = \left( 0.5 - 0.094 \frac{D}{H} \right) \cdot H = \text{ - mm}$$

Since  $D/H > 1.333$ ,  $X_i = 7,762.50 \text{ mm}$

The height from the bottom of the Tank Shell to the center of action of the lateral seismic forces related to the **convective liquid force** for ringwall moment,

$$X_c = \left( 1.0 - \frac{\cosh \left( \frac{3.67 H}{D} \right) - 1}{\frac{3.67 H}{D} \sinh \left( \frac{3.67 H}{D} \right)} \right) \cdot H = 12,722.55 \text{ mm}$$

### 7.2.3 OVERTURNING MOMENT

The seismic overturning moment at the base of the tank shell shall be the SRSS summation of the impulsive and convective components multiplied by the respective moment arms to the center of action of the forces.

Ringwall moment,

$$M_{rw} = \sqrt{[A_i (W_i \cdot X_i + W_s \cdot X_s + W_r \cdot X_r)]^2 + [A_c (W_c \cdot X_c)]^2} = 3.81453E+11 \text{ Nmm}$$
$$= 381453029.8 \text{ Nm}$$

### 7.2.4 SHEAR FORCE

The seismic base shear shall be defined as the SRSS combination of the impulsive and convective components.

$$V = \sqrt{V_i^2 + V_c^2} = 48,326,902.75 \text{ N}$$

where,  $V_i = A_i (W_s + W_r + W_f + W_i) = 47,902,181.05 \text{ N}$   
 $V_c = A_c \cdot W_c = 6,393,010.26 \text{ N}$

### 7.3 RESISTANCE TO OVERTURNING

#### 7.3.1 THICKNESS OF THE BOTTOM PLATE UNDER THE SHELL & ITS RADIAL WIDTH

Bottom/Annular plate thickness, $t_a$	=	16.00 mm
Thickness of bottom shell course, $t_s$	=	28.00 mm
Bottom/Annular plate radial width, $L_s$	=	1200.0 mm
Min. specified yield strength of bottom annulus, $F_y$	=	241.0 N/mm <sup>2</sup>
Min. specified yield strength of bottom shell course, $F_{ty}$	=	241.0 N/mm <sup>2</sup>

Anchorage Ratio,  $J$

$$J = \frac{M_{rw}}{D^2 (W_t (1 - 0.4 A_v) + W_a)} = 2.17$$

where,

$A_v$ =	Vertical earthquake acceleration coefficient	=	0.7
$W_t$ =	Tank and roof weight acting at base of shell	=	28.24 N/mm
$w_a$ =	Resisting force of the annulus	=	94.93 N/mm

Weight of tank shell and portion of roof supported by the shell,

$$W_t = \frac{W_s}{\pi \cdot D} + w_{rs} = 28.24 \text{ N/mm}$$

$$w_{rs} = \text{Roof load acting on the shell, including 10\% of specified snow load. ( Zero for floating roof)} = 0 \text{ N/mm}$$

The resisting force of the annulus,

$$w_a = 99 t_a \sqrt{F_y \cdot H \cdot G_e} \leq 196 \cdot H \cdot D \cdot G_e = 94,932.54 \text{ N/m}$$

$$w_a < 196 \cdot H \cdot D \cdot G_e = 114,016,732,704.00$$

$$G_e = \text{Effective specific gravity including vertical seismic effect} = G \cdot (1 - 0.4 A_v) = 0.72$$

Since the anchorage ratio,  $J > 1.54$ , the tank is not stable and cannot be self-anchored for the design load. The tank shall be mechanically anchored.

### 7.3.2 ANNULAR PLATE REQUIREMENT

If the thickness of the bottom plate under the shell is thicker than the remainder of the bottom, then the minimum radial width of the bottom plate,

$$L = 0.01723 t_a \sqrt{\frac{F_y}{H \cdot G_e}} = 1,108.57 \text{ mm}$$

$$\text{The maximum width of annulus for determining the resisting force, } 0.035 D = 1,366.09 \text{ mm}$$

$$\text{Since } L < 0.035 D, \text{ the minimum radial width should be } = 1,108.57 \text{ mm}$$

And,

$$\text{Since } L_s > L, \text{ the bottom/ annular plate width is } \text{ satisfactory.}$$

### 7.3.3 SHELL COMPRESSION

MECHANICALLY-ANCHORED TANKS

Maximum longitudinal shell compression,

$$\sigma_c = \left( w_t (1 + 0.4 A_v) + \frac{1.273 M_{rw}}{D^2} \right) \frac{1}{t_s} = 12.67 \text{ N/mm}$$

### 7.3.4 MAXIMUM ALLOWABLE SHELL COMPRESSION

$$A = \frac{GHD^2}{t_s^2} \quad (D \text{ in } m) = 40.223 \text{ m}^3/\text{mm}^2$$

For  $GHD^2/(t_s^2) < 44 \text{ m}^3/\text{mm}^2$ ,

$$F_c = \frac{83 \cdot t_s}{2.5D} + 7.5 \{G \cdot H\}^{1/2} = 57.94 \text{ N/mm}^2$$

For  $GHD^2/(t_s^2) \geq 44 \text{ m}^3/\text{mm}^2$ ,

$$F_c = \frac{83 \cdot t_s}{D} = - \text{ N/mm}^2$$

$$\text{Therefore, } F_a (< 0.5 F_y) = 57.94 \text{ N/mm}^2$$

$$\text{Since } \sigma_c < F_c, \text{ therefore the tank is structurally } \text{ stable.}$$

## 7.4 FREE BOARD FOR SLOSHING WAVE HEIGHT

Sloshing wave height above the product design height,

$$\delta_s = 0.5 D. A_f = 1,647.06 \text{ mm}$$

where:

**For SUG I and II,**

**When  $T_c \leq 4$**

$$A_f = K. SD_1. I. \left( \frac{1}{T_c} \right) = 2.5 K Q Fa So I \left( \frac{T_s}{T_c} \right) = 0.21$$

**When  $T_c > 4$**

$$A_f = K. SD_1. I. \left( \frac{4}{T_c^2} \right) = 2.5 K Q Fa So I \left( \frac{4T_s}{T_c^2} \right) = 0.13$$

**For SUG III**

**When  $T_c \leq T_L$**

$$A_f = K. SD_1 \left( \frac{1}{T_c} \right) = 2.5 K Q Fa So \left( \frac{T_s}{T_c} \right) = 0.14$$

**When  $T_c > T_L$**

$$A_f = K. SD_1 \left( \frac{T_L}{T_c^2} \right) = 2.5 K Q Fa So \left( \frac{T_s. T_L}{T_c^2} \right) = 0.08$$

Since SUG is **III** and  **$T_c > T_L$** ,  $A_f = 0.08$

For  $S_{DS} = Q Fa S_s = 0.9 > 0.33g$ ,  
Minimum required freeboard,  $\delta_{req}$  (as per Table E-7) = 1,647.06 mm

## 7.5 TANK ANCHORAGE

### 7.5.1 GEOMETRIC DATA

Number of bolts, N	=	86
Dia. of anchor bolt, d	=	64 mm
Dia. of anchor bolt, d, corr (less c.a.= 3.000 mm) (min.size.25.4 mm)	=	58 mm
Bolts circle diameter, Da	=	39,320 mm
Root area of each hold down bolt, Ab	=	2,642 mm <sup>2</sup>
Spacing between anchor bolts, Sp	=	1,436 mm

### 7.5.2 MATERIAL & MECHANICAL PROPERTIES

Material used	:	SA 320 Gr L7
Specific minimum yield stress, Sy	=	551.5 N/mm <sup>2</sup>
Allowable tensile strength, St.all ( 0.80Sy ) ( Table 5-21a )	=	441.20 N/mm <sup>2</sup>

Uplift force due to seismic loading,

$$W_{AB} = \left( \frac{1.273 Mrw}{Dc^2} - wt ( 1 - 0.4 Av ) \right) + w_{int} = 36,592,019 \text{ N}$$

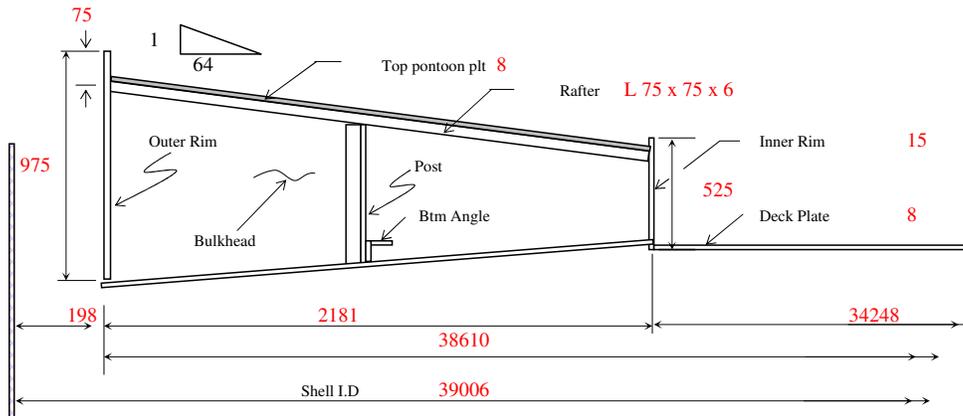
where

Mrw = Overturing moment due to seismic	=	3.81453E+11 Nmm
Dc = Nominal diameter of tank	=	39,031 mm
wt = Tank and roof weight acting at base of shell,	=	28.24 N/mm
Av = Vertical earthquake acceleration coefficient	=	0.70
w <sub>int</sub> = Uplift thrust due to internal pressure	=	0 N/mm

Tensile stress,  
 $\sigma_b = W_{AB} / N.A_b = 161.04 \text{ N/mm}^2$

Since  $\sigma_b < St.all$ , therefore the anchor bolt size is **satisfactory**.

## 8.0 DESIGN OF SINGLE DECK FLOATING ROOF FOR A STORAGE TANK



( All dimensions in mm unless otherwise stated. )

### 8.1 TANK GEOMETRY DATA

Inside diameter , Di ( corroded ) (@	39,000	mm )	=	39,006	mm
Tank height (tan/tan), H			=		
Material of Construction			=	SA 516 Gr 65N	
Specific Minimum Yield Stress, Sy			=	275	N/mm <sup>2</sup>
Modulus of Elasticity			=	209,000	N/mm <sup>2</sup>
Density of Material, ρ (plate)			=	7,850	kg/m <sup>3</sup>
Corrosion Allowance			=	3	mm
Min. Specific Gravity of product			=	0.7	
Max. Specific Gravity of product			=	1	

### 8.2 GEOMETRY DATA

Outer Rim Height, Hor	=	975	mm
Inner Rim Height, Hir	=	525	mm
Pontoon width, w	=	2181	mm
Rim Gap	=	198	mm
Outer Rim Extend above pontoon, Hext	=	75	mm
No. of pontoons, N	=	22	
Outer Rim Diameter, Øor	=	38610	mm
Inner Rim Diameter, Øir	=	34248	mm
Bulkhead Outer heigh, Boh	=	884	mm
Bulkhead Inner heigh, Bih	=	509	mm
Bulkhead Width, wb	=	2157	mm

### 8.3 MEMBER SIZE & PROPERTIES

Outer Rim Thk, Tor	=	9	mm
Inner Rim Thk, Tir	=	15	mm
Top Pontoon Thk, Ttp	=	8	mm
Btm Pontoon Thk, Tbp	=	8	mm
Bulkheads Thk, Tb	=	8	mm
Deck Plate Thickness, Td	=	8	mm
Circumferential Truss Plates	=	8	mm
Rafter	44 Nos. of	L 75 x 75 x 6	@ unit weight of 6.85 kg/m
Posts	44 Nos. of	L 75 x 75 x 6	@ unit weight of 6.85 kg/m

8.4 ROOF SUPPORT LEG (Refer to Design of Supporting Legs)

8.4.1 PONTOON LEG

No. of Pontoon Leg, Np			=	22
Pontoon Leg Size	3" pipe x Sch.	80	@ unit wt	15.27 kg/m
Pontoon Leg Housing	4" pipe x Sch.	80	@ unit wt	22.32 kg/m
Pontoon Leg length			=	2940 mm
Pontoon Leg Housing length			=	1084 mm

8.4.2 DECK LEG

No. of Deck Leg, Nd	(Area of deck / 30m <sup>2</sup> / leg)		=	30
Deck Leg Size	3" pipe x Sch.	80	@ unit wt	15.27 kg/m
Deck Leg Housing	4" pipe x Sch.	80	@ unit wt	22.32 kg/m
Deck Leg length			=	2927 mm
Deck Leg Housing length			=	823 mm

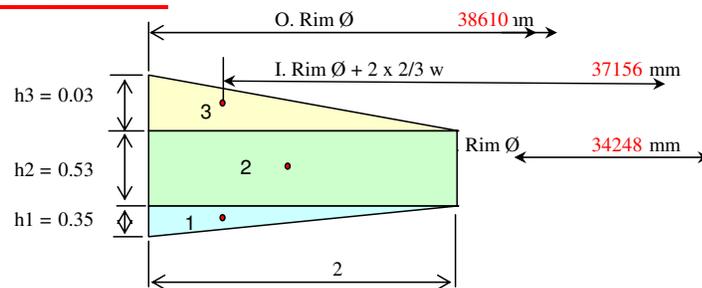
8.5 WEIGHT CALCULATION

Top Pontoon	=	$\pi/4 \times (\phi_{or}^2 - \phi_{ir}^2) \times T_{tp} \times \rho(\text{plate})$	=	15,675.18 kg
Bottom Pontoon	=	$\pi/4 \times (\phi_{or}^2 - \phi_{ir}^2) \times T_{bp} \times \rho(\text{plate})$	=	15,675.18 kg
Inner Rim	=	$\pi \times \phi_{ir} \times H_{ir} \times T_{ir} \times \rho$	=	6,651.28 kg
Outer Rim	=	$\pi \times \phi_{or} \times H_{or} \times T_{or} \times \rho$	=	8,355.38 kg
Bulkheads	=	$1/2 \times (\text{Boh} - \text{Bih}) \times w_b \times T_b \times \rho \times N$	=	2,075.65 kg
Deck Plate	=	$\pi/4 \times \phi_{ir} \times T_d \times \rho$	=	57,852.21 kg
Pontoon Legs	=		=	987.66 kg
Pontoon Legs housing	=		=	532.29 kg
Deck Legs	=		=	1340.86 kg
Deck Legs housing	=		=	551.08 kg

**TOTAL WEIGHT**

Pontoon Components: -	(W <sub>pontoon</sub> )	=	55,248.45 kg
Deck Components: -	(W <sub>deck</sub> )	=	57,852.21 kg
<b>Total Weight of Floating Roof, (W<sub>roof</sub>)</b>		=	<b>113,100.66 kg</b>

9.0 PONTOON VOLUME



Volume 1	=	40.70 m <sup>3</sup>
Volume 2	=	120.17 m <sup>3</sup>
Volume 3	=	3.85 m <sup>3</sup>
<b>Total Pontoon Volume, Vol(pontoon)</b>	=	<b>164.72 m<sup>3</sup></b>

9 .0 **SETTING DECK LEVEL**

9 .1 **OPERATION FLOATATION LEVEL - DECK**

$$\frac{\text{Deck Floation Depth}}{\text{Deck Thk}} = \frac{\text{Density of Deck}}{\text{Density of Product}}$$

$$\text{Floation Depth, } D_{(\text{deck})} = \frac{\rho (\text{deck})}{\rho (\text{product})} \times Td = 89.71 \text{ mm}$$

9 .2 **OPERATION FLOATATION LEVEL - PONTOON**

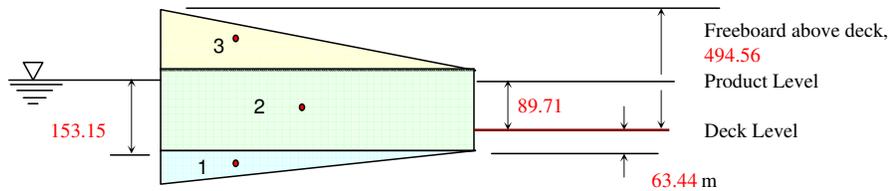
$$\begin{aligned} \text{Buoyant Force, } FB &= F_{\text{pontoon}} \\ \rho \times V_{\text{displacement}} \times g &= W (\text{Pontoon}) \times g \end{aligned}$$

$$\text{Product Displacement, } V_{\text{displacement}} = \frac{\text{Pontoon Weight, } W_{(\text{pontoon})}}{\rho (\text{product})} = 78.93 \text{ m}^3$$

To find Floation Depth of Pontoon from Inner Corner of Pontoon,

$$D_{(\text{pontoon})} = \frac{\text{Vol. Displacement above Inner corner of Pontoon}}{\text{Pontoon Cross Area in Vol. 2}}$$

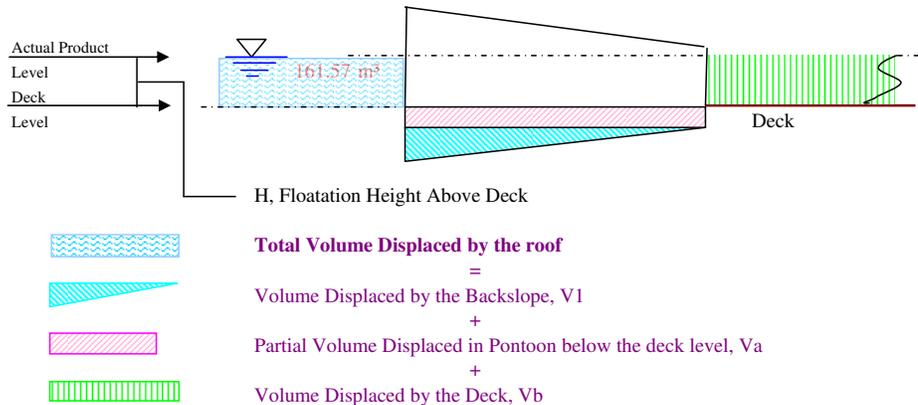
$$D_{(\text{pontoon})} = \frac{V_{\text{displacement}} - V_{\text{backslope (Vol.1)}}}{\frac{1}{4} \times \pi \times (\phi_{\text{or}}^2 - \phi_{\text{ir}}^2)} = 153.15 \text{ mm}$$



The Deck is set at the difference of floation depth in Pontoon & Deck,

$$D_{(\text{deck})} - D_{(\text{pontoon})} = 63.44 \text{ mm}$$

9 .3 **NORMAL OPERATION FLOATATION LEVEL FOR ROOF - PONTOON & DECK**



Total Volume Displaced by the roof,  $V_{\text{displacement (roof)}}$ :

$$V_{\text{displacement (roof)}} = \frac{\text{Roof Total Weight, } W_{(\text{roof})}}{\rho (\text{product})} = 161.57 \text{ m}^3$$

i) Volume Displaced by the Backslope, Volume 1  = 40.70

ii) Partial Volume Displaced in Pontoon below the deck level:  
$$\frac{\text{Deck level Height, h}}{\text{Bulk head outer height, Bih}} \times \text{Vol. 2} = 14.98 \text{ m}^3$$

iii) Volume Displaced by the Deck:  
$$\frac{\text{Area of Deck Plate} \times \text{Floatation Height Above Deck}}{\pi/4 \times \text{Øir}^2 \times H} = 921.21 H$$

Hence, The Floatation Height Above Deck, H = 0.11 m  
114.95 mm

#### 9 4 FLOATATION LEVEL FOR ROOF - PONTOON & DECK FOR 10" (254MM) OF ACCUMULATED RAIN WATER

For deck to support 10" (254mm) of rain water:

Volume of rain water collected at the deck, Vrain =

$$V_{\text{rain}} = A_{\text{deck}} \times H_{\text{rain}} = 233.99 \text{ m}^3$$

where

$$A_{\text{deck}} = \text{Area of deck} = \pi/4 \times \text{Øir}^2 = 921,213,536.64 \text{ mm}^2$$

$$H_{\text{rain}} = \text{Rain accumulation of 10"} = 254.00 \text{ mm}$$

Total Volume Displaced by the roof with the 10" of rain water accumulation, Vdisplacement (rain):

$$V_{\text{displacement (rain)}} = \frac{W(\text{roof}) + W(\text{rain})}{\rho (\text{product})} = 495.84 \text{ m}^3$$

where

W(roof) = Total weight of roof

W(rain) = Weight of 10" rain water

Floatation Height above Deck,

$$H(\text{rain}) = \frac{V_{\text{displacement (rain)}} - \text{Vol.1} - \text{partial of Vol.2 (ii)}}{\text{Area of roof}} = 0.38 \text{ m}$$

$$= 375.95 \text{ mm}$$

#### 10 0 CHECKING THE STRESSES AND DEFLECTION IN THE CENTRE DECK

(Ref. to Roark's Formulas For Stress And Strain, 7th Edition)

#### 10 1 CASE 1: NORMAL CASE - NO PONTOON PUNCTURED

$$\frac{q \alpha^4}{Et^4} = K_1 \frac{y}{t} + K_2 \left( \frac{y}{t} \right)^3 \quad (11.11.1)$$

$$\frac{\sigma \alpha^2}{Et^2} = K_3 \frac{y}{t} + K_4 \left( \frac{y}{t} \right)^2 \quad (11.11.2)$$

Where:

t = Plate thickness, Deck (mm) = Td = 8

$\alpha$  = Outer radius of the deck plate = Øir / 2 = 17124

q = Unit lateral pressure (equiv. weight of deck that float on product)

= Td x (  $\rho(\text{plate}) - \rho(\text{product})$  ) = 0.000561 N/mm<sup>2</sup>

y = Maximum deflection

$\sigma_b$  = bending stress

$\sigma_d$  = diaphragm stress

$\sigma$  =  $\sigma_b + \sigma_d$  = Maximum stress due to flexure and diaphragm tension combined

v = Poisson's ratio = 0.3

E = Modulus of Elasticity = 209,000 N/mm<sup>2</sup>

The deck plate is fixed and held at its outer edge by the pontoon, hence condition is consider as:  
**Fixed and Held. Uniform pressure q over entire plate (Case 3 in Roark's Formulas)**

$$K_1 = \frac{5.33}{1 - \nu^2} = 5.86$$

$$K_2 = \frac{2.6}{1 - \nu^2} = 2.86$$

At the Centre,

$$K_3 = \frac{2}{1 - \nu} = 2.86$$

$$K_4 = 0.976$$

At the edge,

$$K_3 = \frac{4}{1 - \nu^2} = 4.40$$

$$K_4 = 1.73$$

For  $\frac{q \alpha^4}{Et^4} = 56,361.13$

And

$$K_1 \frac{y}{t} + K_2 \left( \frac{y}{t} \right)^3 = \frac{q \alpha^4}{Et^4} = 56,249.31$$

$$y = 215.81 \text{ mm}$$

Solving equation 11.11.2

$$\frac{\sigma \alpha^2}{E \cdot t^2} = K_3 \left( \frac{y}{t} \right) + K_4 \left( \frac{y}{t} \right)^2$$

$$= 787.3494954 \quad (\text{at Deck Center})$$

$$= 1377.567315 \quad (\text{at Deck Edge})$$

At Deck Center,

$$\sigma_{\text{total}} = 35.92 \text{ N/mm}^2$$

$$\sigma_{\text{bending}} = 3.52 \text{ N/mm}^2$$

$$\sigma_{\text{diaphragm}} = 32.40 \text{ N/mm}^2$$

At Deck Edge,

$$\sigma_{\text{total}} = 62.84 \text{ N/mm}^2$$

$$\sigma_{\text{bending}} = 5.41 \text{ N/mm}^2$$

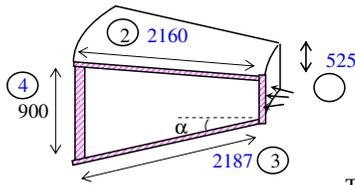
$$\sigma_{\text{diaphragm}} = 57.43 \text{ N/mm}^2$$

It is the diaphragm stress at the edge which causes the tension at the outer edge of the Deck.  
Hence, the radial force on the inner rim,

$$R_h = \sigma_{\text{diaphragm}} \times \text{deck thickness} = 459.44 \text{ N/mm}$$

10.2 **PONTOON STRESS DESIGN - CASE 1**

10.2.1 **PONTOON PROPERTIES**



Nominal diameter of Inner Rim,  $\phi_{ir}$  = 34248 mm  
 Pontoon Inside Width = 2160 mm  
 Inner Rim Thickness,  $T_{ir}$  = 12 mm  
 Outer Rim Thickness,  $T_{or}$  = 9 mm  
 Top Pontoon Thk,  $T_{tp}$  = 8 mm  
 Btm Pontoon Thk,  $T_{bp}$  = 8 mm

Top Pontoon slope angle @ 1 : 64 = 0.02 rad  
 Backslope angle,  $\alpha$  = 0.16 rad

	A (mm <sup>2</sup> )	Y (mm)	AY (mm <sup>3</sup> )	h (mm)	A.h <sup>2</sup> (mm <sup>4</sup> )	I = (bd <sup>3</sup> )/12 (mm <sup>4</sup> )
1	6300	6	37,800	1,126	7,980,578,762	75,600
2	17282	1092	18,872,063	40	26,969,435	6,720,924,525
3	17494	1092	19,103,800	40	27,300,602	6,971,562,462
4	8100	2176.5	17,629,650	1,045	8,845,340,202	54,675
<b>TOTAL</b>	<b>49,176</b>		<b>55,643,313</b>		<b>16,880,189,001</b>	<b>13,692,617,263</b>

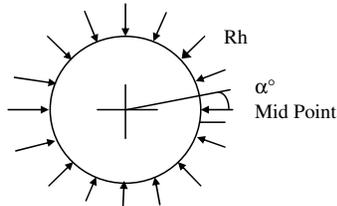
Neutral axis of combined section, C1 = 1132 mm  
 Moment of inertia of section,  $I_{x-x}$  = 30,572,806,264 mm<sup>4</sup>  
 Section modulus available,  $Z_a$  = 27,019,626 mm<sup>3</sup>

10.2.2 **MATERIAL PROPERTIES**

Material Properties : SA 516 Gr. 65N  
 Specified minimum yield stress,  $S_y$  = 275.00 N/mm<sup>2</sup>  
 Yield strength reduction factor,  $k$  ( Table M-1 ) = 1.000  
 Allowable stress reduction factor ( App. M.3.5 ),  $K_s$  ( =  $k.S_y/206.7$  ) = 1.00  
 Allowable bending stress,  $F_b$  = 183.33 N/mm<sup>2</sup>  
 Allowable compressive stress,  $F_c$  = 165.00 N/mm<sup>2</sup>

10.2.3 **PONTOON RING DESIGN**

The uniform radial force acting on the Inner Rim is modelled as load point at each mm of circumference, with a very small angle between load point approximated to uniform distributed load in the circular ring design.



Number of load point @ each mm,  
 $N_{lp} = \pi \times \phi_{ir} = 107,593.27$   
 Angle  $\alpha^\circ = 1/2 \times 360 / N_{lp} = 0.001673^\circ$   
 Radial load on rim,  $R_h = 459.44$  N  
 ( Note :  $R_h$  is negative for inward force )

(Reference to Roark's Formulas For Stress and Strain, 7th Edition, Table 9.2 Case 7)

At Mid-Point,

Bending moment,  

$$M_m = \frac{R_h \cdot D_o}{4} \left( \frac{1}{\sin \alpha} - \frac{1}{\alpha} \right)$$

Circ. tensile force,  

$$T_m = \frac{R_h}{2 \cdot \sin \alpha}$$

At Reaction-Point,

Bending moment,  

$$M_r = - \frac{R_h \cdot D_o}{4} \left( \frac{1}{\alpha} - \frac{1}{\tan \alpha} \right)$$
  
 (  $D_o = \phi_{ir}$ , nominal diameter of inner ring )

Circ. tensile force,  

$$T_r = \frac{R_h}{2 \tan \alpha}$$

10 .2.4 RESULT

RING STABILITY CHECK		MID-POINT	LOAD-POINT
Bending Moment	( Nmm )	19.14	-38.29
Circumferential force	( N )	7,867,429	7,867,429
Bending Stress	( N/mm <sup>2</sup> )	0.0000007	-0.000001
Circumferential stress	( N/mm <sup>2</sup> )	159.98	159.98
Allow. bending stress	( N/mm <sup>2</sup> )	183	183.33
Allow. axial stress	( N/mm <sup>2</sup> )	165	165
Unity Check		0.97	0.97
Condition		<b>OK.</b>	<b>OK.</b>

10 .3 CASE 2: INFLUENCE OF 10" (254mm) OF RAIN ACCUMULATED ON CENTER DECK



For deck to support 10" (254mm) of rain water:

Volume of rain water collected at the deck,

$$V_{\text{rain}} = A_{\text{deck}} \times H_{\text{rain}} = 233.99 \text{ m}^3$$

where

$$A_{\text{deck}} = \text{Area of deck} = \pi/4 \times \varnothing_{\text{ir}}^2 = 921,213,536.64 \text{ mm}^2$$

$$H_{\text{rain}} = \text{Rain accumulation of 10"} = 254 \text{ mm}$$

$$\text{Weight of 10" accumulated rain water, } W_{\text{rain}} = \text{Vol. rain} \times \rho_{\text{rain}} = 233,988.24 \text{ kg}$$

$$\text{Upward Bouyant Load} = \text{Deck Area} \times \text{Floatation Height} \times \text{Product density}$$

$$= \pi/4 \times (\varnothing_{\text{ir}})^2 \times H_{\text{(rain)}} \times \rho = 242,429.27 \text{ kg}$$

$$\text{Downward load due to deck steel and rain water,} = 291,840.45 \text{ kg}$$

$$= W_{\text{deck}} + W_{\text{rain}}$$

Nett downward force acting on deck =

$$= \frac{(\text{Upward bouyant load} - \text{Downward Load})}{\text{Deck Area}} = 53.64 \text{ kg/m}^2$$

75

$$\frac{q \alpha^4}{Et^4} = K_1 \frac{y}{t} + K_2 \left( \frac{y}{t} \right)^3 \quad (11.11.1)$$

$$\frac{\sigma \alpha^2}{Et^2} = K_3 \frac{y}{t} + K_4 \left( \frac{y}{t} \right)^2 \quad (11.11.2)$$

Where:

- t = Plate thickness, Deck (mm) = Td = 8
- α = Outer radius of the deck plate = Ø<sub>ir</sub> / 2 = 17124
- q = Unit lateral pressure = 0.000526 N/mm<sup>2</sup>
- y = Maximum deflection
- σ<sub>b</sub> = bending stress
- σ<sub>d</sub> = diaphragm stress
- σ = σ<sub>b</sub> + σ<sub>d</sub> = Maximum stress due to flexure and diaphragm tension combined
- v = Poisson's ratio = 0.3
- E = Modulus of Elasticity = 200,000 N/mm<sup>2</sup>

The deck plate is fixed and held at its outer edge by the pontoon, hence condition is consider as:

Case 3 - Fixed and Held. Uniform pressure q over entire plate

$$K_1 = \frac{5.33}{1 - \nu^2} = 5.86$$

$$K_2 = \frac{2.6}{1 - \nu^2} \quad K_3 = \frac{2}{1 - \nu} = 2.86$$

At the Centre,

$$K_3 = \frac{2}{1 - \nu} = 2.86$$

$$K_4 = 0.976$$

At the edge,

$$K_3 = \frac{4}{1 - \nu^2} = 4.40$$

$$K_4 = 1.73$$

For  $\frac{q \alpha^r}{Et^3} = 55,228.70$

And

$$K_1 \frac{y}{t} + K_2 \left( \frac{y}{t} \right)^3 = \frac{q \alpha^r}{Et^3} = 55,140.73$$

$$y = 214.38325 \text{ mm}$$

Solving equation 11.11.2

$$\begin{aligned} \frac{\sigma \alpha^2}{E \cdot t^2} &= K_3 \left( \frac{y}{t} \right) + K_4 \left( \frac{y}{t} \right)^2 \\ &= 777.4581306 \quad (\text{at Deck Center}) \\ &= 1360.154003 \quad (\text{at Deck Edge}) \end{aligned}$$

At Deck Center,

$$\begin{aligned} \sigma_{\text{total}} &= 33.94 \text{ N/mm}^2 \\ \sigma_{\text{bending}} &= 3.34 \text{ N/mm}^2 \\ \sigma_{\text{diaphragm}} &= 30.60 \text{ N/mm}^2 \end{aligned}$$

At Deck edge,

$$\begin{aligned} \sigma_{\text{total}} &= 59.37 \text{ N/mm}^2 \\ \sigma_{\text{bending}} &= 5.14 \text{ N/mm}^2 \\ \sigma_{\text{diaphragm}} &= 54.23 \text{ N/mm}^2 \end{aligned}$$

It is the diaphragm stress at the edge which causes the tension at the outer edge of the Deck.

Hence, the radial force on the inner rim,

$$R_h = \sigma_{\text{diaphragm}} \times \text{deck thickness} = 433.85 \text{ N/mm}$$

10.4 **PONTOON STRESS DESIGN - CASE 2**

10.4.1 **PONTOON PROPERTIES**

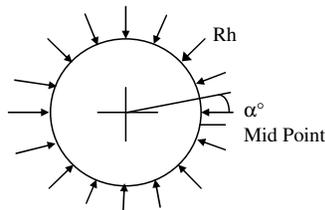
Nominal diameter of Inner Rim, Ø <sub>ir</sub>	=	34248 mm
Section modulus available, Z <sub>a2</sub> =	=	27019626.01 mm <sup>3</sup>
Cross sectional area, A <sub>a</sub>	=	49,176 mm <sup>2</sup>

10.4.2 **MATERIAL PROPERTIES**

Material Properties	=	SA 516 Gr. 65N
Specified minimum yield stress, S <sub>y</sub>	=	275.00 N/mm <sup>2</sup>
Yield strength reduction factor, k ( Table M-1 )	=	1.000
Allowable stress reduction factor ( App. M.3.5 ), K <sub>s</sub> ( = k.S <sub>y</sub> /206.7 )	=	1.00
Allowable bending stress, F <sub>b</sub>	=	183.33 N/mm <sup>2</sup>
Allowable compressive stress, F <sub>c</sub>	=	165.00 N/mm <sup>2</sup>

10.4.3 **PONTOON RING DESIGN**

The uniform radial force acting on the Inner Rim is modelled as load point at each mm of circumference, with a very small angle between load point approximated to uniform distributed load in the circular ring design.



Number of load point @ each mm,  
 $N_{lp} = \pi \times \text{Ø}_{ir} = 107593.27$   
 Angle  $\alpha^\circ = 1/2 \times 360 / N_{lp} = 0.001673^\circ$   
 Radial load on rim, Rh = 433.85 N/ load pt  
 ( Note : Rh is negative for inward force )

(Reference to Roark's Formulas For Stress and Strain, 7th Edition, Table 9.2 Case 7)

At Mid-Point,

Bending moment,  

$$M_m = \frac{Rh \cdot Do}{4} \left( \frac{1}{\sin \alpha} - \frac{1}{\alpha} \right)$$

Circ. tensile force,  

$$T_m = \frac{Rh}{2 \cdot \sin \alpha}$$

At Reaction-Point,

Bending moment,  

$$M_r = \frac{Rh \cdot Do}{4} \left( \frac{1}{\alpha} - \frac{1}{\tan \alpha} \right)$$

Circ. tensile force,  

$$T_r = \frac{Rh}{2 \tan \alpha}$$

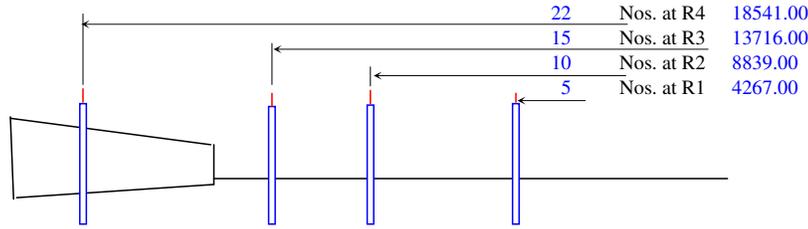
10.4.4 **RESULT**

RING STABILITY CHECK		MID-POINT	LOAD-POINT
Bending Moment	( Nmm )	18.08	-36.15
Circumferential force	( N )	7,429,209	7,429,209
Bending Stress	( N/mm <sup>2</sup> )	0.0000007	-0.0000001
Circumferential stress	( N/mm <sup>2</sup> )	151.07	151.07
Allow. bending stress	( N/mm <sup>2</sup> )	183	183
Allow. axial stress	( N/mm <sup>2</sup> )	165	165
Unity Check		0.92	0.92
Condition		<b>OK.</b>	<b>OK.</b>

10.4.5 **STRESSES SUMMARY**

	LOAD CASE 1		LOAD CASE 2	
	Deck Center	Deck Edge	Deck Center	Deck Edge
σ <sub>total</sub> ( N/mm <sup>2</sup> )	35.92	62.84	33.94	59.37
σ <sub>bending</sub> ( N/mm <sup>2</sup> )	3.52	5.41	3.34	5.14
σ <sub>diaphragm</sub> ( N/mm <sup>2</sup> )	32.40	57.43	30.60	54.23

**11 .0 ROOF SUPPORT LEG DESIGN**



**11 .1 GEOMETRIC DATA**

Support leg size	= 3" Sch. 80	
Pipe outside diameter	= 88.9	mm
Pipe Thickness,	= 7.62	mm
Pipe Area, $A_{leg}$	= 1,945.76	mm <sup>2</sup>
Radius of gyration, $r =$	$\sqrt{\frac{I}{A_{leg}}}$ $\sqrt{\frac{Do2 - Di2}{4}}$	= 24.89

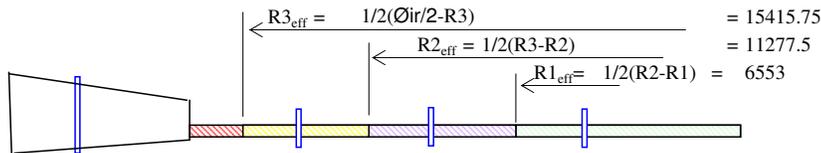
**11 .2 MATERIAL PROPERTIES**

Material of Construction for roof support leg	: SA 333 Gr 6
Specific Minimum Yield Stress, $S_y$	= 241 N/mm <sup>2</sup>
Modulus of Elasticity	= 209,000 N/mm <sup>2</sup>
Density of Material, $\rho$ (plate)	= 7,850 kg/m <sup>3</sup>
Leg Material	

**11 .3 LOADING DATA**

Support leg length at		
i) R1 : Lsp1	= 2927	mm
ii) R2 : Lsp2	= 2927	mm
iii) R3 : Lsp3	= 2927	mm
iv) R4 : Lsp4	= 2940	mm
Deck O.D	= 34231	mm
Deck Thickness, $t_d$	= 8	mm
Deck Area, $A_{deck}$	= 920,299,220.87	mm <sup>2</sup>
Center deck weight, $W_{deck}$	= 57,794.79	kg
Design Live Load, $L_{live}$	= 1.2	KN/m <sup>2</sup>

Effective radius for area of deck supported by leg:



Area of deck supported by legs at

i) $R1 = \pi(R1_{eff})^2$	= 134,905,671.69	mm <sup>2</sup>
ii) $R2 = \pi((R2_{eff})^2 - (R1_{eff})^2)$	= 264,648,384.82	mm <sup>2</sup>
iii) $R3 = \pi((R3_{eff})^2 - (R2_{eff})^2)$	= 347,030,823.13	mm <sup>2</sup>
iv) $R4 = \pi((\text{Ø}_{deck})^2 - (R3_{eff})^2)$	= 173,714,341.24	mm <sup>2</sup>

#### 11 .4 SUPPORT LEG AT INNER DECK R1

No. of legs at R1	=	5
Area of deck supported by legs at R1, A1	=	134,905,671.69 mm <sup>2</sup>
Deck area on each leg, A1'	=	26,981,134.34 mm <sup>2</sup>
Deck load on one leg = $W_{deck} \times \frac{A1'}{A_{deck}}$	=	1,694.42 kg
	=	16.62 KN
Live load on one leg = $L_{live} \times A1'$	=	32.38 KN
Total load on one leg = Deck load + Live load	=	49.00 KN
Stress on support leg at inner deck R1, P1 = Total Load / A <sub>leg</sub>	=	25.18 N/mm <sup>2</sup>

#### 11 .4.1 ALLOWABLE STRESS

As per AISC code, Slenderness ratio, $\lambda = K.Lsp1 / R_{x-x}$	=	118
where K	=	1
Column slenderness ratio dividing elastic and inelastic buckling, $Cc = \sqrt{\frac{2\pi^2 E}{S_y}}$	=	130.84
When $\lambda \leq Cc$ , $Sc.all = \frac{[1 - \lambda^2 / 2Cc^2] S_y}{5/3 + 3\lambda / 8Cc - \lambda^3 / 8Cc^3}$ (i)	=	75.08 N/mm <sup>2</sup>
When $Cc \leq \lambda \leq 120$ , $Sc.all = \frac{12\pi^2 E}{23 \lambda^2}$ (ii)	=	77.80 N/mm <sup>2</sup>
When $120 \leq \lambda \leq 200$ , $Sc.all = \frac{\text{Smaller of (i) or (ii)}}{1.6 - \lambda/200}$	=	74.20 N/mm <sup>2</sup>
In this case, the allowable stress Sc.all is	=	75.08 N/mm <sup>2</sup>
Since P1 < Sc.all, the support leg at inner deck R1 is <b>satisfactory</b> .		

#### 11 .5 SUPPORT LEG AT INNER DECK R2

No. of legs at R2	=	10
Area of deck supported by legs at R2, A2	=	264,648,384.82 mm <sup>2</sup>
Deck area on each leg, A2'	=	26,464,838.48 mm <sup>2</sup>
Deck load on one leg = $W_{deck} \times \frac{A2'}{A_{deck}}$	=	1,661.99 kg
	=	16.30 KN
Live load on one leg = $L_{live} \times A2'$	=	31.76 KN
Total load on one leg = Deck load + Live load	=	48.06 KN
Stresses on support leg at inner deck R2, P2 =	=	24.70 N/mm <sup>2</sup>

#### 11 .5.1 ALLOWABLE STRESS

As per AISC code, Slenderness ratio, $\lambda = K.Lsp2 / R_{x-x}$	=	118
where K	=	1
Column slenderness ratio dividing elastic and inelastic buckling, $Cc = \sqrt{\frac{2\pi^2 E}{S_y}}$	=	130.84

When  $\lambda \leq C_c$ ,

$$Sc.all = \frac{[1 - \lambda^2 / 2C_c^2] \cdot S_y}{5/3 + 3\lambda / 8C_c - \lambda^3 / 8C_c^3} \quad (i) = 75.08 \text{ N/mm}^2$$

When  $C_c \leq \lambda \leq 120$ ,

$$Sc.all = \frac{12\pi^2 E}{23 \lambda^2} \quad (ii) = 77.80 \text{ N/mm}^2$$

When  $120 \leq \lambda \leq 200$ ,

$$Sc.all = \frac{\text{Smaller of (i) or (ii)}}{1.6 - \lambda / 200} = 74.20 \text{ N/mm}^2$$

In this case, the allowable stress  $Sc.all$  is = **75.08 N/mm<sup>2</sup>**

Since  $P2 < Sc.all$ , the support leg at inner deck R2 is **satisfactory**.

#### 11 .6 SUPPORT LEG AT INNER DECK R3

No. of legs at R3 = 15

Area of deck supported by legs at R3,  $A_3$  = 347,030,823.13 mm<sup>2</sup>

Deck area on each leg,  $A_3'$  = 23,135,388.21 mm<sup>2</sup>

Deck load on one leg =  $W_{deck} \times \frac{A_3'}{A_{deck}}$  = 1,452.90 kg

Live load on one leg =  $L_{live} \times A_3'$  = 14.25 KN

Total load on one leg = Deck load + Live load = 27.76 KN

Stresses on support leg at inner deck R3,  $P_3$  = Total Load /  $A_{leg}$  = **21.59 N/mm<sup>2</sup>**

#### 11 .6.1 ALLOWABLE STRESS

As per AISC code,  
Slenderness ratio,  
 $\lambda = K \cdot L_{sp3} / R_{x-x} = 118$   
where  
 $K = 1$   
Column slenderness ratio dividing elastic and inelastic buckling,

$$C_c = \sqrt{\frac{2\pi^2 E}{S_y}} = 130.84$$

When  $\lambda \leq C_c$ ,

$$Sc.all = \frac{[1 - \lambda^2 / 2C_c^2] \cdot S_y}{5/3 + 3\lambda / 8C_c - \lambda^3 / 8C_c^3} \quad (i) = 75.08 \text{ N/mm}^2$$

When  $C_c \leq \lambda \leq 120$ ,

$$Sc.all = \frac{12\pi^2 E}{23 \lambda^2} \quad (ii) = 77.80 \text{ N/mm}^2$$

When  $120 \leq \lambda \leq 200$ ,

$$Sc.all = \frac{\text{Smaller of (i) or (ii)}}{1.6 - \lambda / 200} = 74.20 \text{ N/mm}^2$$

In this case, the allowable stress  $Sc.all$  is = **75.08 N/mm<sup>2</sup>**

Since  $P_3 < Sc.all$ , the support leg at inner deck R3 is **satisfactory**.

**11 .7 SUPPORT LEG AT PONTOON**

No. of legs at R4 = 27

Area of deck supported by legs at R4, A4 = 173,714,341.24 mm<sup>2</sup>

Deck area on each leg, A4' = 6,433,864.49 mm<sup>2</sup>

Deck load on one leg =  $W_{deck} \times \frac{A4'}{A_{deck}}$  = 404.05 kg

Pontoon weight, W<sub>pontoon</sub> = 3.96 KN

Pontoon weight on one leg, W<sub>pontoon'</sub> = 55,248.45 kg

Live load on one leg = L<sub>live</sub> x A4' = 49.27 KN

Total load on one leg = Deck load + Live load + Pontoon weight = 7.72 KN

Stresses on support leg at Pontoon, P4 = Total Load / A<sub>leg</sub> = 31.33 N/mm<sup>2</sup>

**11 .7.1 ALLOWABLE STRESS**

As per AISC code,  
Slenderness ratio,  
 $\lambda = K.L_{sp4} / R_{x-x}$  = 118

where  
K = 1

Column slenderness ratio dividing elastic and inelastic buckling,  
 $Cc = \frac{\sqrt{2\pi^2 E}}{Sy}$  = 130.84

When  $\lambda \leq Cc$ ,  
 $Sc.all = \frac{[1 - \lambda^2 / 2Cc^2].Sy}{5/3 + 3\lambda / 8Cc - \lambda^3 / 8Cc^3}$  (i) = 74.62 N/mm<sup>2</sup>

When  $Cc \leq \lambda \leq 120$ ,  
 $Sc.all = \frac{12\pi^2 E}{23 \lambda^2}$  (ii) = 77.12 N/mm<sup>2</sup>

When  $120 \leq \lambda \leq 200$ ,  
Smaller of (i) or (ii)  
 $Sc.all = \frac{1.6 - \lambda / 200}{1.6 - \lambda / 200}$  = 73.93 N/mm<sup>2</sup>

In this case, the allowable stress Sc.all is = 74.62 N/mm<sup>2</sup>

Since P3 < Sc.all, the support leg at inner deck R3 is satisfactory.

**11 .8 STRESSES SUMMARY**

Leg at radius	No. of leg	Actual stress, (N/mm2)	Allowable stress, (N/mm2)	RESULT
4267.00	5.00	25.18	75.08	OK
8839.00	10.00	24.70	75.08	OK
13716.00	15.00	21.59	75.08	OK
18541.00	22.00	31.33	74.62	OK

## BLEEDER VENT CALCULATION

### 12 .0 DESIGN OF AIR VENTING SYSTEM

#### 12 .1 GEOMETRIC DATA

Design Code	:	API STD 2000
Inside diameter, Di	=	39000 mm
Tank height, H	=	20700 mm
Nominal Capacity	=	24000 m <sup>3</sup>
Design pressure, Pi	=	2.50 mbarg
Flash point (FP)/Normal boiling point (NBP) (@ FP )	=	67 °C
Filling rate ( Pumping in/Flow rate to tank ), Vi	=	427 m <sup>3</sup> /hr
Emptying rate ( Pumping out/Flow rate from tank ), Vo	=	1,100 m <sup>3</sup> /hr

#### OPERATING VENTING

### 12 .2 NORMAL VACUUM VENTING

#### 12 .2.1 Maximum liquid movement out of a tank

Flow rate of free air, Vv1 ( = Vo/15.9 x 15.89 )	=	1097.23 m <sup>3</sup> /hr
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#### 12 .2.2 Thermal inbreathing

Tank capacity, V	=	155,535 barrels
From Table 2, column 2 (Thermal Venting Capacity Req't), Flow rate of free air, Vv2 ( @ 0 ft <sup>3</sup> /hr )	=	0 m <sup>3</sup> /hr
Total vacuum flow required, Vv ( = Vv1 + Vv2 )	=	1,097 m <sup>3</sup> /hr

### 12 .3 NORMAL PRESSURE VENTING

#### 12 .3.1 Maximum liquid movement into a tank

Rate of free air per 0.159m <sup>3</sup> /hr of product import rate, m	=	0.17 m <sup>3</sup> /hr
Flow rate of free air, Vp1 ( = Vi/0.159 x m )	=	457 m <sup>3</sup> /hr

#### 12 .3.2 Thermal outbreathing

From Table 2, column 3 (Thermal Venting Capacity Req't), Flow rate of free air, Vp2 ( @ 0 ft <sup>3</sup> /hr )	=	0 m <sup>3</sup> /hr
Total pressure flow required, Vp ( = Vp1 + Vp2 )	=	457 m <sup>3</sup> /hr

#### OPEN VENT SIZING ( BLEEDER VENT SIZING )

### 12 .4 OPEN VENT SIZING CALCULATION

Maximum flow, Q ( @ Vacuum flow at ( @ 2.50 mbarg. )	=	1,097 m <sup>3</sup> /hr
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$$Q = K \cdot A \cdot \sqrt{2 \cdot g \cdot H}$$

where

K = Discharge coefficient	=	0.62
A = cross sectional area of vent		
g = acceleration due to gravity		
H = Head as measure pressure differential		

$$H = \frac{\Delta p}{\gamma} = 21 \text{ m}$$

Minimum require cross sectional area of vent,

$$A_{v\_req} = \frac{Q}{K \sqrt{2 \cdot g \cdot H}} = \frac{Q}{K \sqrt{2 \cdot g \cdot \Delta p}} = \begin{matrix} 0.0241 \text{ m}^2 \\ 24,124 \text{ mm}^2 \end{matrix}$$

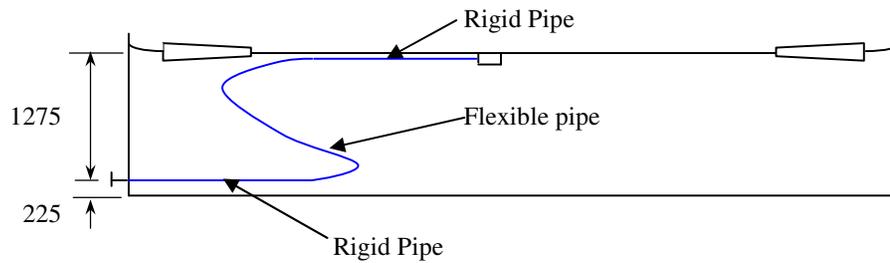
where

Q = Max. Air flow required	=	0.3048 mm <sup>3</sup> /s
γ = Specific weight of Air = ρ g	=	11.812 kg/m <sup>2</sup> s <sup>2</sup>
ρ = Air density	=	1.204 kg/m <sup>3</sup>
Δp = Differential pressure	=	250 N/m <sup>2</sup>

### 12 .5 BLEEDER VENT SELECTED

Selected bleeder vent size	:	8" Sch Std
Number of vent, N	=	1
Outside diameter of the vent, do	=	219
Inside Dia. of one vent , di ( @ vent pipe thickness = 8.18 mm )	=	202.64 mm
Total cross sectional area of vents, Av_actual	=	32,251 mm <sup>2</sup>
Since Av_actual > Ar_gnv, therefore the nos. & size of vents is		satisfactory.

### 13 .0 ROOF DRAIN DESIGN



### 13 .1 GEOMETRIC DATA

Tank Nominal Diameter	=	39,000 mm
Tank Height,	=	20,100 mm
Roof lowest height, H	=	1500 mm
Drain outlet nozzle elevation, z	=	225 mm
Roof Deck Area	=	920.30 m <sup>2</sup>
Design Rain Fall	=	50 mm/hr
Design drainage required, Qreq.	=	46.01 m <sup>3</sup> /hr
No. of Roof Drain, N	=	2
Roof drain pipe size (rigid & fitting)	=	4" Sch 80
Dain Pipe Outside Diameter, Do	=	101.6 mm
Drain pipe thickness	=	8.56 mm
Drain Pipe length :		
L1 = Rigid	20 m x	2 nos. = 40 m
L2 = Flexible	23.14 m x	1 nos. = 23.14 m

### 13 .2 Number of Fitting & Accessories per drain pipe

- 45° elbow	N <sub>45°</sub>	=	2
- 90° elbow	N <sub>90°</sub>	=	1
- Valve	N <sub>v</sub>	=	1
- Rigid pipe		=	2
- Flexible pipe		=	1

### 13 .3 TOTAL HEAD

$$H = h + \frac{V^2}{2g}$$

### 13 .4 TOTAL HEAD LOSS OF ROOF DRAIN PIPE

$$h = \frac{V^2}{2g} \times \frac{K L'}{D}$$

Where

H =	Total head between the lowest position of deck and the roof drain nozzle	=	1.275	m
G =	Gravity acceleration			
K =	Friction Coefficient			
	- For rigid pipe :	K <sub>1</sub>	=	0.0168
	- For flexible pipe :	K <sub>2</sub>	=	0.03
L' =	Total equivalent length of drain pipe			
D =	Inside Diameter of drain pipe	=	0.08448	m

### 13 .5 EQUIVALENT PIPE LENGTH OF VALVE AND FITTING

Accordance to NFPA 15 Table 8.5.2.1,

Equivalent length for	4" 45° elbow, L <sub>45°</sub>	=	3.1
	90° elbow, L <sub>90°</sub>	=	1.2
	Valve, L <sub>v</sub>	=	0.6

Total equivalent pipe length for **RIGID PIPE**:

$$L_1' = L_1 + N_{45^\circ} \times L_{45^\circ} + N_{90^\circ} \times L_{90^\circ} + N_v \times L_v = 48 \text{ m}$$

Total equivalent pipe length for **Flexible PIPE**:

$$L_2' = L_2 = 23.14 \text{ m}$$

### 13 .6 TOTAL HEAD LOSS OF ROOF DRAIN PIPE

$$h = \frac{V^2}{2g} \times \left( \frac{K_1 L_1'}{D} + \frac{K_2 L_2'}{D} \right)$$

$$H = \frac{V^2}{2g} \left( \frac{K_1 L_1'}{D} + \frac{K_2 L_2'}{D} + 1 \right)$$

### 13 .7 FLOW VELOCITY

$$V = \sqrt{\frac{2 g H}{\left( \frac{K_1 L_1'}{D} + \frac{K_2 L_2'}{D} + 1 \right)}} = 1.15 \text{ m/s}$$

### 13 .8 DRAINAGE FLOW RATE PER DRAIN PIPE

$$Q = \text{AREA} \times \text{Velocity}$$

$$= \pi/4 \times D^2 \times V \times 3600 \text{ (s/hr)} = 23.30 \text{ m}^3 / \text{hr}$$

### 13 .9 MINIMUM ROOF DRAIN REQUIRED

$$N_{req} = \frac{\text{Drainage flow rate required}}{\text{Actual flow rate per drain}} = 1.97$$

**MINIMUM REQUIRED = 2**

**14 WEIGHT ANALYSIS**

**ITEM NO :** 7061T-3901

<b>1</b>	<b>GENERAL</b>		Type of roof support :		Type of roof	
	Design code :	API 650 11th Edition	NA		: Floating Roof	
	Inside diameter :	39,000 mm	Tank height :		20,700 mm	
	Steel density		Roof plates lapping factor :		Annular/Bottom plates lapping factor : 1	
<b>2</b>	<b>SHELL COURSES</b>					
	<b>ONE - FOOT METHOD (OUTER TANK)</b>					<b>Y</b>
	<b>Course No.</b>	<b>Material</b>	<b>Thickness (mm)</b>	<b>Width (mm)</b>	<b>Weight (kg)</b>	
	1	A 516 GR. 65N	28.00	2,440	65,757	
	2	A 516 GR. 65N	25.00	2,440	58,707	
	3	A 516 GR. 65N	22.00	2,440	51,658	
	4	A 516 GR. 65N	19.00	2,440	44,611	
	5	A 516 GR. 65N	16.00	2,440	37,564	
	6	A 516 GR. 65N	13.00	2,440	30,518	
	7	A 516 GR. 65N	11.00	2,020	21,377	
8	A 516 GR. 65N	11.00	2,020	21,377		
9	A 516 GR. 65N	11.00	2,020	21,377		
10	-	-	-	-		
Total weight of shell plates =					352,948 kg	
<b>3</b>	<b>BOTTOM PLATES</b>					
	<b>Material</b>		<b>Thickness (mm)</b>	<b>Outside Dia. (mm)</b>	<b>Weight (kg)</b>	
A 516 GR. 65N			9.00	39,130	84,961	= 84,961 kg
<b>4</b>	<b>TOP CURB ANGLE</b>					
	<b>Material</b>	<b>Size</b>	<b>Qty</b>	<b>Length (mm)</b>	<b>Unit Weight (kg/m)</b>	<b>Weight (kg)</b>
A 516 GR. 65N		76 x 76 x 6.4	1	122,827	10.33	1,269 = 1,269 kg
<b>5</b>	<b>TOP WIND GIRDERS</b>					
	<b>Material</b>	<b>Size</b>	<b>Qty</b>	<b>Length (mm)</b>	<b>Unit Weight (kg/m)</b>	<b>Weight (kg)</b>
A 516 GR. 65N		T 825 x 250 x 8 x 1	1	125,183	87.51	10,955 = 10,955 kg
<b>6</b>	<b>INTERMEDIATE WIND GIRDERS</b>					
	<b>Material</b>	<b>Size</b>	<b>Qty</b>	<b>Length (mm)</b>	<b>Unit Weight (kg/m)</b>	<b>Weight (kg)</b>
A 516 GR. 65N		T 405 x 150	1	124,476	53.76	6,691 = 6,691 kg
<b>7</b>	<b>NOZZLES</b>					
	Total weight of nozzles					1,500
<b>8</b>	<b>MISCELLANEOUS</b>					
	Assuming	5.00	% of total weight			22,916 = 22,916 kg
<b>9</b>	<b>STAIRWAY &amp; PERIMETER PLATFORM</b>					
	Platform Weight	165.00	KN			16,820 = 16,820 kg
<b>10</b>	<b>OPERATING LIQUID WEIGHT</b>					
	Operating liquid height	(@ =	20,700	mm & sg @ =		1.00 ) = 24,728,026 kg
<b>11</b>	<b>HYDROSTATIC WATER WEIGHT</b>					
	Hydrostatic water height	(@	20,700	mm )		= 24,728,026 kg
<b>ERECTION WEIGHT (Exclude roof)</b>					= 498,060 kg	
<b>OPERATING WEIGHT</b>					= 25,226,086 kg	
<b>FIELD HYDROSTATIC TEST WEIGHT</b>					= 25,226,086 kg	